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Vierendeel girders : stress analysis, design and welding,

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INTRODUCTION

The collapse of the Hasselt bridge in 1938 aroused the interest within the British structural and welding engineers and during the last war, coming into contact with some of them, we talked the matter over on several occasions. (1).

I seized the opportunity to draw their attention on Professor Baes' researches and method of calculation (2).

I was able to show them what Professor Despretts, at that time Chief of the Bridges Designing Office of the Belgian National Railways, had done in the field of riveted Vierendeel bridges (see photos of some of his works).

I stressed the particular care he took over the conceiving and detail work of double track Vierendeel bridges at Malines, Liége and Herenthals. (3)

The present note was used as base to our expression of views.

I hope it may revive the interest of the structural and welding engineers in a truss girder for which its author, Professor Vierendeel, fought with such eagerness during many years.

Be it said incidentally, I was able to draw the attention on Professor Merten's theorem regarding the derivative of strain energy with respect to any one load or couple. Merten's paper published in the *Annales des Ingénieurs sortis de Gand*, 1908, gives Castigliano's theorems their generalized meanings and does not seem to be well known abroad.

Bridge structures of this type have

(1) I wish to thank Mr. MacMardo, Chief Engineer of the L. M. S. Railway, Glasgow, for the help he kindly gave me at that time

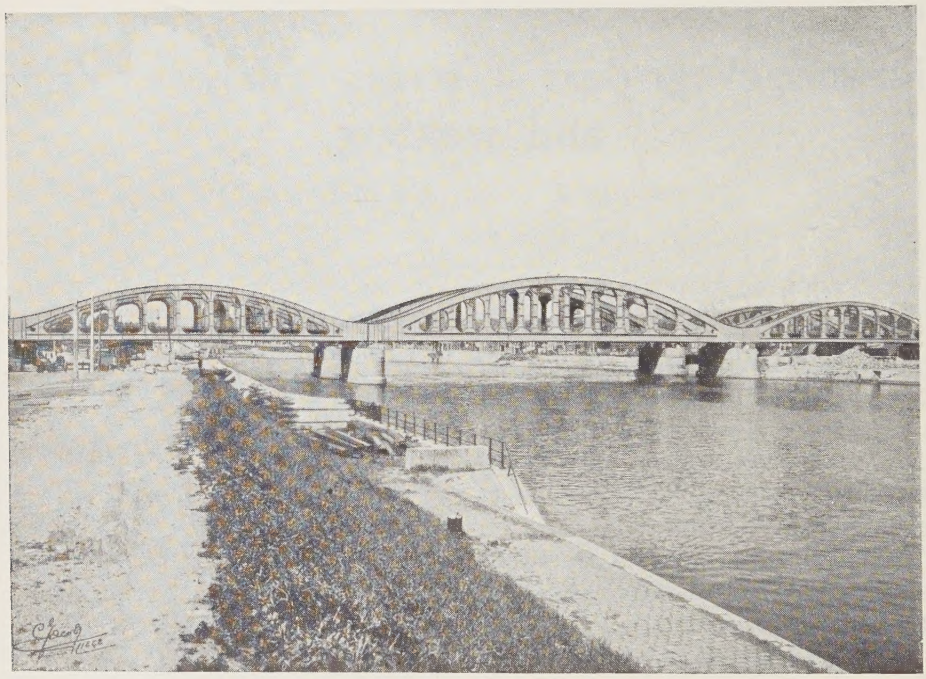
(2) Professor Baes' extensive researches have been published in the Belgian review *L'Ossature Métallique* before 1940.

(3) For Vierendeel Railway bridges, see more particularly a paper published by Professor Despretts in the *Bulletin* of our Association, January 1936.

been built to a considerable extent in Belgium and in the Belgian Congo.

They have also been used elsewhere in Europe, namely in Poland and in Czechoslovakia, in the United States of America and in Japan. Such frames are known as quadrangular girders, open-web girders and industrial district. Among these 66 bridges, 51 were steel bridges of which 26 belonged to the Vierendeel type.

All the Vierendeel girders on the Albert Canal have been designed with a parabolic top chord, the bottom chord being straight and resting freely on two bearings.



Designer : Professor DESPRETS

Photo 1. — Vierendeel railway bridge over the Meuse at Liège.

or trusses or Vierendeel girders, after Professor VIERENDEEL of the University of Louvain.

This type of girder has been used on a large scale for the crossing of the Albert Canal. 66 bridges have been built, nearly one every 1 mile 1/4 so as not to impede in the future the dense traffic on the important road system of this mining

There are in my view three types of arch ribs with tie rods or tie girders.

1. The arch rib with tie rod, i. e. the bowstring girder, the common case is the roof arch rib, a tie rod being provided between the springings.

In this case, the arch rib has a section and a moment of inertia sufficient to balance the axial load, the whole of the

bending and the shear. The abutments are helped by the tie rod which takes in tension the arch thrust.

2. In the last few years, bridges have been built where the conception about the action of arch rib and tie girder is just the opposite.

The tie girder is usually an important built up plate girder capable of balancing the axial load and all the bending and shear. The arch rib has only a stiffening action.

(3) A third type, very often used in France for the bridges in ferroconcrete is an intermediate solution between these two extreme first conceptions.

The top and bottom chords have nearly the same moment of inertia and the bottom chord takes a part of the bending and of the shear.

The Vierendeel truss belongs to this third type. As you know economy is claimed for this type of girder, Professor Vierendeel being able to increase the allowable stress in comparison with the primary stresses admitted in ordinary triangular steel trusses.

He does so on the ground that the ordinary truss is subjected to secondary stresses in addition to the primary stresses.

But this is not the point I wish to debate. As I told you 26 Vierendeel bridges were built over the Albert Canal, they looked quite in harmony with their surroundings, nicely shaped and light. One, the Hasselt bridge collapsed in 1938 and was the subject of researches and discussions in the technical world.

I will later point out and allow myself to comment briefly on the views of the Institute of Welding, views with which I quite agree.

Let us remember that the majority of of the Vierendeel girders under review were welded.

Since the collapse of the Hasselt bridge, two more bridges, the Herenthals-Oolen bridge and the Kaulille bridge were closed to traffic in January 1940; wide cracks leading to the complete breaking of the bottom chord occurred when the temperature was very low (— 20° C, — 4° F) (See photos 2 and 3 and the 2 accompanying figures.)

The decks were composed of a system of cross girders and stringers covered with a slab in ferro concrete. This slab had been laid with transverse joints, out and out cuts as required by the staff of the Belgian Royal Engineers.

The consequence was of course an abrupt change in the moment of inertia of the bottom chord and deck.

It should be pointed out that in both cases, the breaking of the bottom chord occurred in the panels where the cuts in the ferro concrete slabs were located.

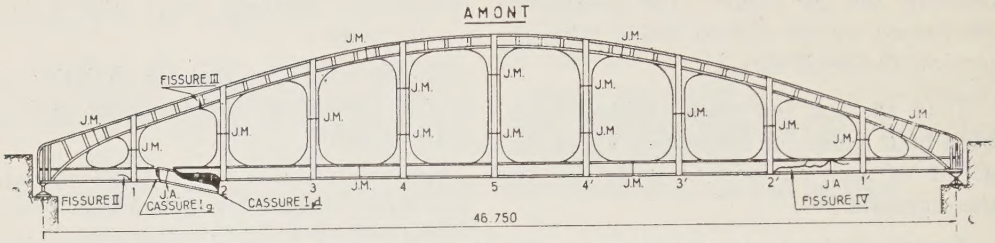
The deck and the stringers might have prevented the complete collapse of both bridges.

In the transactions of the *Institute of welding*, January 1940, no. 1, it is said about the Hasselt bridge :

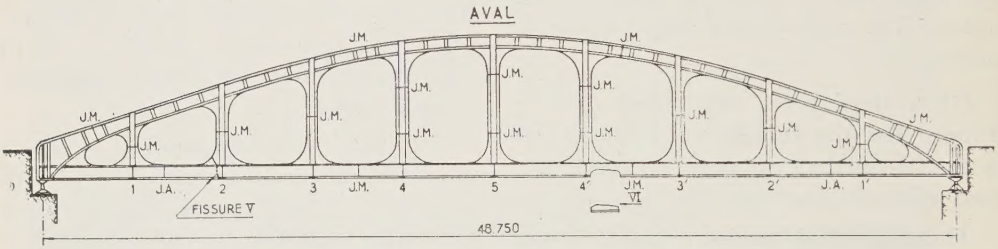
« The failure must have been due to one or more of the following causes :

» *a*) overstressing either generally or locally due to incorrect design. »

I would like to show you how these girders can be calculated and to discuss the connection of a vertical post to a chord ; in my opinion this connection



View in elevation of the girder (up-stream) of the Kaulille bridge.



View in elevation of the girder (down-stream) of the Kaulille bridge.

Fissure = crack. — Cassure = break.

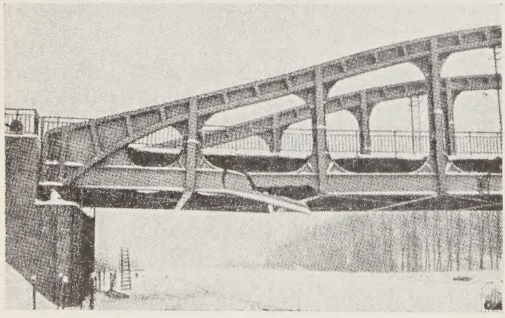


Photo 2. — Breaks : bottom chord.

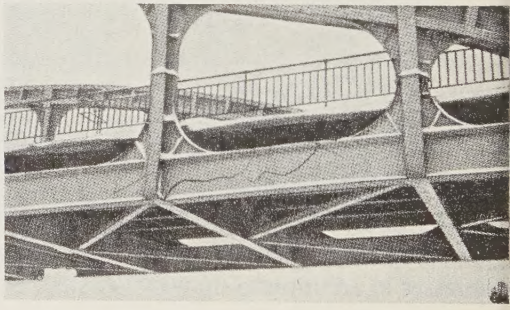


Photo 3. — Cracking : bottom chord.

ought to be improved as it is a weak point of the welded structure.

The Hasselt bridge failure has been for the Belgian welding engineers a turning point in this new technique of structural engineering.

I would like at the end of this expression of views to show the results reached in that field when war broke out in 1940.

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A freely supported Vierendeel girder resting on two bearings is what I would call an internally statically indeterminate structure.

If I cut the top chord, for instance, at one section. I must introduce on both sides of the cut an axial load N_r, a shear T_r and a bending moment M_r to preserve the equilibrium of the girder.

Each quadrangular panel, let us say each mesh if cut through the top chord will give three similar unknown reactions.

If there are m meshes, we will get 3 m unknown internal reactions which we will determine in writing 3 m equations of deformation expressing the rotation and the displacements along two axes of the different cuts (method of Bresse and of Vierendeel, applied by the latter to the vertical posts cut at the inflection points). Better, it is only a question of personal liking, I will apply the principle of least work.

According to this principle, as we all know, the stresses in such a frame are such that the total internal work or resilience is a minimum.

But to solve all these equations is a long, tedious process and mistakes are easily made.

It is, therefore, quite natural that those who studied the Vierendeel girder tried to simplify the problem by taking into account the position in the vertical posts of the points of inflection or contraflexure. So did first Professor VIERENDEEL.

When he tested in 1897 an openweb girder at Tervueren near Brussels, he went so far, by increasing the load, as to get these points of inflection in some posts.

Numerous tests by photoelastic methods carried out in the last few years by Professor Baes of Brussels University have proved the existence of these points of inflection and more important, have shown how their location varies according to the relative moments of inertia of the chords and the posts.

We had the opportunity of attending Professor Baes' lectures on this subject and got the impression that the problem is now solved for the designer. Important papers have been published by Mr. BAES, and his pupils. They have established the equations of the isoclinics (loci of points of equal orientation of principal stresses), of the isochromatics (loci of maximum shearing stress) and of the isostatics (envelopes of the principal stresses); all these curves agree with the tests.

These experiments have led to an important practical conclusion : in some cases, the assumptions previously made were wrong and could be misleading.

And now, how are we going to calculate a Vierendeel girder?

As far as I am concerned, I know at least half a dozen methods.

Vierendeel, Keelhoff, Magnel, Saliger, Dana Young, Takabeya, Fransen, Kriso, Baes, de Marneffe, Campus, all have tackled the problem.

By the way, let us point out that HARDY CROSS in his book *Continuous frames of reinforced concrete* is applying to the Vierendeel girder his so powerful method of moment distribution. A great step forward has been made since Professor BAES and his pupils studied and gave the true location of the point of inflection in the posts.

My old professor, Professor KEELHOFF, at Ghent University, some thirty years ago used to locate the point of inflection, h' and h'' being the distances of this point to the top and bottom chords, J', J'', α', α'' the moments of inertia and the slopes

of these chords according to the formula :

$$\frac{h'}{h''} = \frac{J' \cos \alpha'}{J'' \cos \alpha''}$$

pointing out that this relation was not so far wrong if J' was nearly equal to J''.

Usually, the two chords have nearly the same moment of inertia and Keelhoff's method which based on Kriso's idea has been well fixed up by Professor BAES.

I will only show you the principles of it and will take, to simplify the problem, the case of a parallel flange truss, the moments of inertia of the chords being equal $J_r' = J_r''$ and those of the vertical posts constant J.

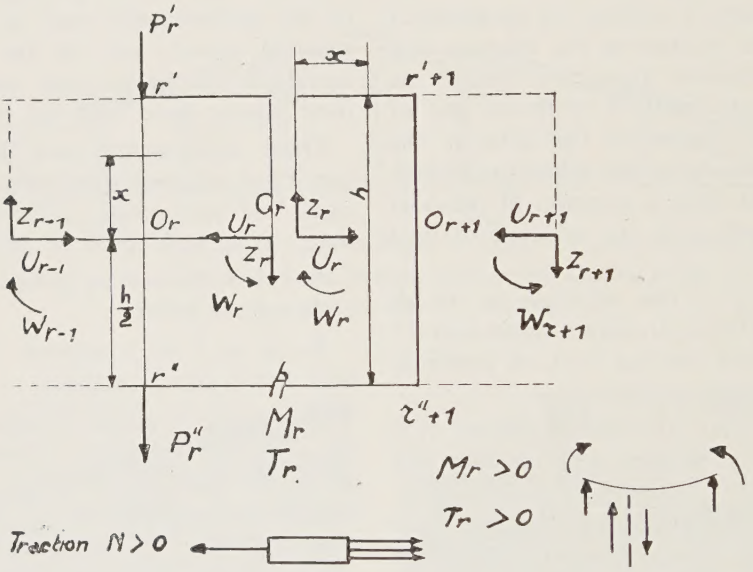

Fig. 1.

Note. — M_r, T_r are due to the externally statically determinate loading.

method, improved by Magnel yields good results.

Professor VIERENDEEL even suggests that we can assume the points of inflection located on top of the posts supposed hinged the moment of inertia of the top chord tending to the value nil, the area remaining constant; many bridges have been calculated on this assumption, some twenty years ago.

I don't want to dwell upon this question just now, I prefer to give you firstly the

Suppose that we know the position of the points of inflection given by the fairly correct formula

$$\frac{h'}{h} = \frac{\frac{b}{h} \cdot \frac{J}{J''} + 6}{\frac{b}{h}\left(\frac{J}{J'} + \frac{J}{J''}\right) + 12}$$

for all the posts, the outside ones excepted for which 6 has to be replaced by 2,5; 12 by 5.

$$\text{If } J' = J'', \frac{h'}{h} = \frac{1}{2}$$

Let us study the notations and rules for positive axial loads, shears and bending moments.

With Fransen, Kriso and Baes, cut the top chord in the middle of a mesh and apply the reactions where the vertical line meets in C_r the line joining the two points of inflection O_r and O_{r+1}.

Adequate couples W_r have to be introduced and let us resolve the resultant force Q_r into two components, the one U_r along the line $O_r\ O_{r+1}$, the other Z_r along the vertical.

We proceed the same way with all the meshes of the truss.

As O_r is a point of contraflexure, the bending moment at this point is equal to nil and we get the equation:

$$W_r - W_{r-1} = (Z_r + Z_{r-1}) \frac{b}{2} \quad . \quad . \quad (1).$$

For O_{r+1} :

$$W_{r+1} - W_r = (Z_{r+1} + Z_r) \frac{b}{2} \quad . \quad . \quad . \quad (2)$$

The work done by a couple in moving through an angle is equal to the product of the moment of the couple into the angle turned through.

Therefore, if a short portion ds of a beam subjected to a bending moment M is caused to change its slope by an amount di, the work done in bending is $\frac{M di}{2}$ because M increases gradually from O to M as $di = \frac{M ds}{EI}$,

Total work done in bending :

$$W = \int \frac{M}{2} \cdot \frac{M ds}{EI} = \int \frac{M^2}{2EI} ds$$

and according to the principle of least work :

$$\frac{dW}{dU_r} = \int \frac{M}{EI} \cdot \frac{dM}{dU_r} \cdot ds = 0$$

$$\frac{dW}{dW_r} = \int \frac{M}{EI} \cdot \frac{dM}{dW_r} \cdot ds = 0$$

$$\frac{dW}{dZ_r} = \int \frac{M}{EI} \cdot \frac{dM}{dZ_r} \cdot ds = 0$$

the relations between the increments of W_r and Z_r, dW_r and dZ_r being found by equations (1) and (2) left hand side of the mesh r : $dW_r = dZ_r \times \frac{b}{2}$ right hand side of the mesh r : $- dW_r = dZ_r \times \frac{b}{2}$.

We see that the three unknown internal reactions of a mesh r : U_r, Z_r, W_r have only an influence upon the bending moment of the four bars, chords and posts of this mesh r and that the bending moment of one of the posts, r for instance, can be expressed in function of U_{r-1} and U_r only.

For the posts, $\frac{dM}{dW_r}$ and $\frac{dM}{dZ_r} = 0$

post r, bending moment $M = -(U_{r-1} - U_r)\,x$

post r_{+1}, bending moment $M = (U_r - U_{r+1})\,x$

I. $$\frac{dW}{dW_r} = 0 \int_0^{\frac{b}{2}} \left(W_r + Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(1 - \frac{2}{b} \cdot x\right) dx$$

$$+ \int_0^{\frac{b}{2}} \left(W_r - Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(1 - \frac{2}{b} \cdot x\right) dx$$

$$- \int_0^{\frac{b}{2}} \left(M_r + T_r \cdot x - W_r - Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(1 - \frac{2}{b} \cdot x\right) dx$$

$$- \int_0^{\frac{b}{2}} \left(M_r - T_r \cdot x - W_r + Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(1 - \frac{2}{b} \cdot x\right) dx = 0$$

$$2W_r \int_0^{\frac{b}{2}} \left(1 - \frac{2}{b}\,x\right) dx - 2\,(M_r - W_r) \int_0^{\frac{b}{2}} \left(1 - \frac{2}{b}\,x\right) dx = 0$$

$$W_r = \frac{1}{2}\,M \quad \ldots\ldots\ldots\ldots\ldots \quad (I)$$

II. $$\frac{dW}{dZ_r} = 0 \int_0^{\frac{b}{2}} \left(W_r + Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(x - \frac{b}{2}\right) dx$$

$$- \int_0^{\frac{b}{2}} \left(W_r - Z_r \cdot x - U_r \cdot \frac{h}{2}\right)\left(x - \frac{b}{2}\right) dx$$

$$- \int_0^{\frac{b}{2}} \left(M_r + T_r \cdot x - W_r - Z_r \cdot x - U_r \cdot \frac{h}{2} \right) \left(x - \frac{b}{2} \right) dx$$

$$+ \int_0^{\frac{b}{2}} \left(M_r - T_r \cdot x - W_r + Z_r \cdot x - U_r \cdot \frac{h}{2} \right) \left(x - \frac{b}{2} \right) dx = 0$$

$$4Z_r \int_0^{\frac{b}{2}} x \left(x - \frac{b}{2} \right) dx - 2\, T_r \cdot \int_0^{\frac{b}{2}} x \left(x - \frac{b}{2} \right) dx = 0$$

$$Z_r = \frac{1}{2}\, T_r \cdot$$

III. $$\frac{dW}{dU_r} = 0 - \frac{2}{J} \int_0^{\frac{h}{2}} \left(U_{r-1} - U_r \right) \cdot x^2 dx + \frac{2}{J} \int_0^{\frac{h}{2}} \left(U_r - U_{r+1} \right) \cdot x^2 dx$$

$$+ \frac{2}{J_r'} \int_0^{\frac{b}{2}} \left(W_r + Z_r \cdot x - U_r \cdot \frac{h}{2} \right) \left(- \frac{h}{2} \right) \cdot dx$$

$$+ \frac{2}{J'_r} \int_0^{\frac{b}{2}} \left(W_r - Z_r \cdot x - U_r \cdot \frac{h}{2} \right) \left(- \frac{h}{2} \right) dx = 0$$

$$- U_{r-1} \cdot \frac{h^3}{12J} + U_r \cdot \frac{h^3}{12J} + U_r \cdot \frac{h^3}{12J} - U_{r+1} \cdot \frac{h^3}{12J} - \frac{W_r \cdot h \cdot b}{J_r'} + U_r \cdot \frac{h^2 b}{2J_r'} = 0$$

$$- U_{r-1} + \left(2 + 6 \cdot \frac{b}{h} \cdot \frac{J}{J'_r} \right) \cdot U_r - U_{r+1} = 6 \cdot \frac{b}{h} \cdot \frac{J}{J'_r} \cdot \frac{M_r}{h}$$

emembering that $W_r = \frac{1}{2} M_r$.

Parallel flange truss. Fundamental equations : Mesh *r*.

Summing up, we get :

$$-U_{r-1} + \left(2 + 6 \cdot \frac{b}{h} \cdot \frac{J}{J'_r}\right) U_r - U_{r+1} = 6 \cdot \frac{b}{h} \cdot \frac{J_r}{J_r'} \cdot \frac{M_r}{h}$$

$$W_r = \frac{1}{2} M_r$$

$$Z_r = \frac{1}{2} T_r$$

Loading of the top chord.

$$\text{Axial load } N_{r'1r'+1} = -U_r$$

$$\text{Shear } T_{r'1r'+1} = \frac{1}{2} T_r$$

$$\text{Bending moment. } (M_{r'1r'+1})\ x = +\frac{1}{2}(M_x - U_r \cdot h)$$

$$(M_{r'1r'+1})x = +W_r + Z_r \cdot x - U_r \times \frac{h}{2}$$

$$= +\frac{1}{2} M_r + \frac{1}{2} T_r \cdot x - U_r \times \frac{h}{2} = +\frac{1}{2}(M_x - U_r \cdot h)$$

Loading of the bottom chord.

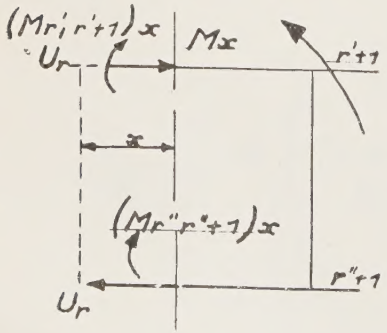

Fig. 2.

$$\text{Axial load } N_{r''1r''+1} = +U_r$$

$$\text{Shear } T_{r''1r''+1} = \frac{1}{2} T_r$$

$$\text{Bending moment } (M_{r''1r''+1})\ x = +\frac{1}{2}(M_x - U_r \cdot h)$$

$$\text{as } (M_{r'1r'+1})x + (M_{r''1r''+1})x = M_x - U_r \cdot h$$

Parallel flange truss.

Fundamental equations taking into account the deformations due to axial loads in the chords, supposed to be identical in sections.

$$W_r = \frac{1}{2} M_r \quad , \quad Z_r = \frac{1}{2} T_r$$

$$- U_{r-1} + \left(2 + 6 \cdot \frac{b}{h} \cdot \frac{J}{J_r'}\right) \cdot U_r - U_{r+1} + 24 \frac{b}{h} \cdot \frac{J}{A_r' \cdot h^2} \cdot U_r = 6 \frac{b}{h} \cdot \frac{J}{J_r'} \cdot \frac{M_r}{h}$$

A_r' = area of chord.

Formerly, we had neglected in the expression of the internal work, the work due to axial loads in the chords :

$$2 \times \frac{1}{2} \frac{N^2 b}{EA_r} \quad , \quad N_r = \pm U_r.$$

Parallel flange truss. Fundamental equations taking into account a variation of temperature, the top chord being at a temperature t', the bottom chord at a temperature t'' and $t' > t''$, k = coefficient of linear expansion.

$$W_{r1t} = 0 \quad Z_{r1t} = 0$$

$$- U_{r-1,t} + \left(2 + 6 \cdot \frac{b}{h} \cdot \frac{J}{J'_r}\right) \cdot U_{r,t} - U_{r+1,t} = 12 \frac{EJ}{h^3} \cdot bk\,(t' - t'')$$

Note that if $t' > t''$, the elongation $b.k\ (t' - t'')$ is measured in opposite direction to the force U_r.

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Rigid frame of a three storied, one aisle building. (1)

Exact analysis of wind stress.

The columns are supposed to be fixed at the ends.

The relative values of the moments of inertia are shown on the figure.

Let us apply the following equations :

$$- U'_{r-1} + \left(1 + \frac{J_r}{J_{r\ 1}} + 6 \frac{b_r}{h} \cdot \frac{J_r}{J_r'}\right) U_r' - \frac{J_r}{J_{r+1}} \cdot U'_{r+1} = 6 \cdot \frac{b_r}{h} \cdot \frac{J_r}{J'_r} \cdot \frac{M_r}{h}$$

$$W_r = \frac{1}{2} M_r \qquad Z_r = \frac{1}{2} T_r$$

(1) Question dealt with at Professor Baes' lectures.

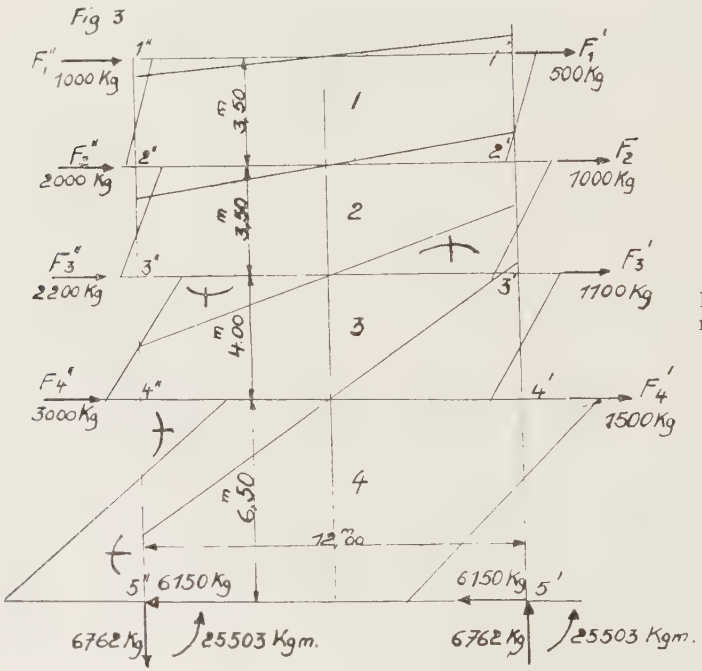
From the load system and the sizes of the frame, we get :

$$W_1 = \frac{1}{2} M_1 = + \ 1\,312{,}5 \text{ kgm.} \qquad Z_1 = \frac{1}{2} T_1 = + \ 750 \text{ kgr.}$$

$$W_2 = \frac{1}{2} M_2 = + \ 6\,562{,}5 \text{ kgm.} \qquad Z_2 = \frac{1}{2} T_2 = + \ 2\,250 \text{ kgr.}$$

$$W_3 = \frac{1}{2} M_3 = + \ 18\,300 \text{ kgm.} \qquad Z_3 = \frac{1}{2} T_3 = + \ 3\,900 \text{ kgr.}$$

$$W_4 = \frac{1}{2} M_4 = + \ 46\,087{,}5 \text{ kgm.} \qquad Z_4 = \frac{1}{2} T_4 = + \ 6\,150 \text{ kgr.}$$



Bending moments.

Fig. 3.

As the columns are fixed in their foundations, the moment of inertia at ground level of the assumed girder is ∞.

The four equations in U are : units : kgr. and meter :

$$+\left(1+\frac{1}{2}+6\times\frac{3.50}{12}\times\frac{1}{1}\right)U'_1-\frac{1}{2}\times U'_2=6\times\frac{3.50}{12}\times\frac{1}{1}\times\frac{2\,625}{12}$$

$$-U'_1+\left(1+\frac{1}{1}+6\times\frac{3.50}{12}\times\frac{2}{1}\right)U'_2-\frac{1}{1}\times U'_3=6\times\frac{3.50}{12}\times\frac{2}{1}\times\frac{13\,125}{12}$$

$$-U'_2+\left(1+\frac{2}{3}+6\times\frac{4}{12}\times\frac{2}{1}\right)U'_3-\frac{2}{3}\times U'_4=6\times\frac{4}{12}\times\frac{2}{1}\times\frac{36\,600}{12}$$

$$-U'_3+\left(1+0+6\times\frac{6.50}{12}\times\frac{3}{2.5}\right)U'_4-0=6\times\frac{6.50}{12}\times\frac{3}{2.5}\times\frac{92\,175}{12}$$

or :

$$\left.\begin{aligned} +\ 3.25\ U'_1 - 0.50\ U'_2 &= 382{,}81\\ -\ U'_1 + 5.50\ U'_2 - U'_3 &= 3\,828{,}12\\ -\ U'_2 + 5.66\ U'_3 - 0.66\ U'_4 &= 12\,200\\ -\ U'_3 + 4.90\ U'_4 &= 29\,956{,}88 \end{aligned}\right\}$$

Thus wet get :

$$\begin{aligned} U'_4 &= +\ 6\,761{,}69 \text{ kgr.}\\ U'_3 &= +\ 3\,175{,}39 \text{ kgr.}\\ U'_2 &= +\ 1\,331{,}90 \text{ kgr.}\\ U'_1 &= +\ 332{,}89 \text{ kgr.} \end{aligned}$$

We could get a rough check by assuming $J_r = J_{r+1} = \cdots = \infty$

$U'_{r\infty} = \dfrac{M_r}{h}$ we get :

$$\begin{aligned} U'_{4\infty} &= +\ 7\,681{,}25 \text{ kgr.}\\ U'_{3\infty} &= +\ 3\,050 \text{ kgr.}\\ U'_{2\infty} &= +\ 1\,093{,}70 \text{ kgr.}\\ U'_{1\infty} &= +\ 218{,}70 \text{ kgr.} \end{aligned}$$

Loading of the girders.

$$\left.\begin{aligned} \text{Axial load} \quad & N_{r'r''} = -\frac{1}{2}(P'_r - P''_r)\\ \text{Shear} \quad & T_{r'r''} = U'_{r-1} - U'_r\\ \text{Bending moment} \quad & (M_{r'r''})_z = (U'_{r-1} - U_r)\times z \end{aligned}\right\}$$

Loading of the columns.

Axial load $N = \pm U'_r$

Shear $T = \frac{1}{2} T_r$

Bending moment $M = + \frac{1}{2} (M_x - U'_r \cdot h)$

	Shear in kgr.	Bending moments at the ends in kgm.	Axial load in kgr.
Girder 1′1″ . . .	$T_{1'1''} = U'_1 = 323$	$323 \times 6 = 1928$	$-\frac{F''_1 - F'_1}{2} = -250$
Girder 2′2″ . . .	$T_{2'2''} = U'_2 - U'_1 = 1009$	$1009 \times 6 = 6054$	$-\frac{F''_2 - F'_2}{2} = -500$
Girder 3′3″ . . .	$T_{3'3''} = U'_3 - U'_2 = 1843$	$1843 \times 6 = 11058$	$-\frac{F''_3 - F'_3}{2} = -550$
Girder 4′4″ . . .	$T_{4'4''} = U'_4 - U'_3 = 3587$	$3587 \times 6 = 21522$	$-\frac{F''_4 - F'_4}{2} = -750$

Columns. Axial load.

$N_{1'2'} = -U'_1 = -323$ kgr. $\quad N_{1''2''} = +323$ kgr.

$N_{2'3'} = -U'_2 = -1\,332$ kgr. $\quad N_{2''3''} = +1\,332$ kgr.

$N_{3'4'} = -U'_3 = -3\,175$ kgr. $\quad N_{3''4''} = +3\,175$ kgr.

$N_{4'5'} = -U'_4 = -6\,762$ kgr. $\quad N_{4''5''} = +6\,762$ kgr.

Shear $T_{1'2'} = T_{1''2''} = Z_1 = 750$ kgr.

$T_{2'3'} = T_{2''3''} = Z_2 = 2\,250$ kgr.

$T_{3'4'} = T_{3''4''} = Z_3 = 3\,900$ kgr.

$T_{4'5'} = T_{4''5''} = Z_4 = 6\,150$ kgr.

Bending moments in kgm.

For the right hand side column 1′. . . . . .5′.

At the top of the stories :

$$(M_{1'2'})_{right} = - Z_1 \times \frac{3.50}{2} - U'_1 \times 6 + W_1 = - 1\,938$$

$$(M_{2'3'})_r = - Z_2 \times \frac{3.50}{2} - U'_2 \times 6 + W_2 = - 5\,367$$

$$(M_{3'4'})_r = - Z_3 \times \frac{4.00}{2} - U'_3 \times 6 + W_3 = - 8\,550$$

$$(M_{4'5'})_r = - Z_4 \times \frac{6.50}{2} - U'_4 \times 6 + W_4 = - 14\,472$$

At the bottom of the stories :

$$(M_{1'2'})_{left} = - Z_1 \times \frac{3.50}{2} - U'_1 \times 6 + W_1 = + 687$$

$$(M_{2'3'})_l = + Z_2 \times \frac{3.50}{2} - U'_2 \times 6 + W_2 = + 2\,508$$

$$(M_{3'4'})_l = + Z_3 \times \frac{4.00}{2} - U'_3 \times 6 + W_3 = + 7\,050$$

$$(M_{4'5'})_l = + Z_4 \times \frac{6.50}{2} - U'_4 \times 6 + W_4 = + 25\,503$$

To check, use the above formula :

$$M = + \frac{1}{2} (M_x - U'_r \cdot h)$$

$$(M_{4'5'})_l = \frac{1}{2} M_4 + \frac{1}{2} T_4 \times \frac{6.50}{2} - \frac{1}{2} U'_4 \times 12 = + 25\,503$$

***

Vierendeel's method of calculation.

In his course on the stability of structures Professor VIERENDEEL lays down certain general formulae enabling one to calculate the strength of his type of girder, whether having parallel flanges or not. He bases his conclusions on the equations of the statics combined with those relating to deformation.

We propose to re-establish the general formulae involved basing ourselves on the theorems of the derivative of strain energy.

We know that if a group of forces in equilibrium gradually acts upon structure previously free from any influence, such forces in consequence of their projected displacements perform an amount of work T, the derivative of which, with respect to any one among them F, is given by the general expression :

$$\frac{dT}{dF} = \frac{1}{E}\int_0^S \frac{N}{\Omega}\cdot\frac{dN}{dF}\cdot ds + \frac{1}{G}\int_0^S \frac{T}{\Omega}\cdot\frac{dT}{dF}\cdot ds + \frac{1}{G}\int_0^S \frac{T'}{\Omega}\cdot\frac{dT'}{dF}\cdot ds + \frac{1}{E}\int_0^S \frac{M}{I}\cdot\frac{dM}{dF}\,ds + \frac{1}{E}\int_0^S \frac{M'}{I'}\cdot\frac{dM'}{dF}\cdot ds + \frac{1}{G}\int_0^S \frac{M''}{I''}\cdot\frac{dM''}{dF}\cdot ds \quad (1)$$

M. VIERENDEEL sets aside displacements due to direct load; in arriving at the work involved we shall take in account that only which is due to the bending moments, so that the general formula (1) becomes reduced to :

$$\frac{dT}{dF} = \frac{1}{E}\int_0^S \frac{M}{I}\cdot\frac{dM}{dF}\cdot ds \quad (2)$$

The first theorem of CASTIGLIANO teaches us that, provided that the force in question F is independent of all other forces or couples we have

$$\frac{dT}{dF} = y_F$$

MERTEN has rendered this theorem of general significance and made it easier to apply it; he has shown that :

$$\frac{dT}{dF} = y_F + \Sigma y\cdot\frac{dP}{dF}$$

P representing the other forces or couples acting upon the structure, y their displacements, projected or angular, the axes of reference being free to be chosen in any way desired, provided that the derivatives of the second member $\frac{dP}{dF}$ are determined by the equations of the statics, which must themselves always be verified and that indeed whatever may be the supplementary loading that one may be led to add to the original real one.

Let us made a section through a Vierendeel girder with parallel flanges as below :

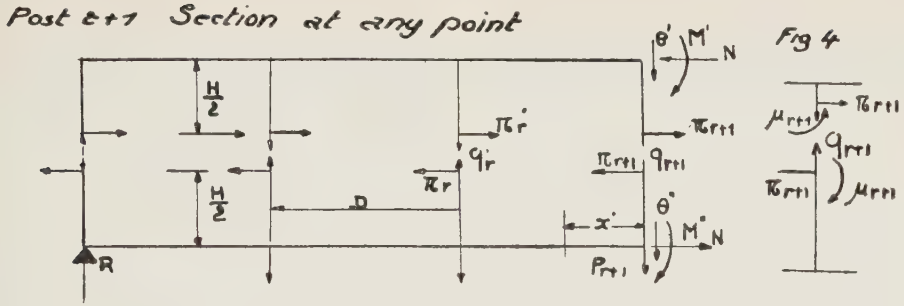


Fig. 4.

Mr. VIERENDEEL assumes that the vertical posts and the flanges have the same moment of inertia I from one end to the other and that a point of inflection is to be found in the middle of the verticals, hence :

$$\mu_{r+1} = 0$$

In consequence of making this section, those originally in contact no longer are so. To bring back the faces of such section to their initial state it is necessary to apply to the two separate sections a symmetrical external effort or, generally speaking, one vertical component q_r and one horizontal π_r and a moment μ_r.

In what follows we shall make use of the notation and conventional signs employed by Professor VIERENDEEL.

If this loading (q_r, π_r, μ_r) has the same value as the elastic stresses developed in the initial structure, the faces of the section will be brought back into contact; if the load is too small, this will not be attained, and if it is too large, the contact will be carried past.

Let us call, as does MERTEN, the *equivalent* loading that corresponding to contact, i. e. equivalent to the elastic stresses developed in the actual non-sectioned piece.

The theorem of minimum work, particular case of those due to CASTIGLIANO or MERTEN, becomes expressed thus :—

« In any hyperstatic system which is sectioned the derivative taken in relation to one of the elements of supplementary loading (q_r, π_r, μ_r) acting thereon, derivative of the work done by the external forces, is nil in the case of the equivalent loading. »

It is clear that if the system, although by its nature hyperstatic, is isostatic in its external reactions, then these reactions are independent of the value of the symmetrical supplementary loading the static effect of which is nil, which is the case of a Vierendeel girder resting freely on two supports.

Let us apply this theorem to the Vierendeel girder with parallel flanges set out above.

If we take into consideration the group of variables π_r π_{r+1} we have :

$$\frac{d\mathrm{T}}{d\,\pi_{r+1}} = 0,$$

the bending moments only being a function of π_{r+1} and π_r in line with the panel r, $r+1$.

The statics give us : $\pi_r + \pi_{r+1} + \Sigma\pi$ (independent of $\pi_{r+1} = 0$ whence

$$\frac{d\pi_r}{d\pi_{r+1}} = -1$$

Then we get :

$$\frac{dT}{d\pi_{r+1}} = 2\int_0^{\frac{H}{2}} (-\pi_r \cdot y) \times y \cdot dy$$

$$+ 2\int_0^{\frac{H}{2}} (-\pi_{r+1} \cdot y) \times (-y) \cdot dy$$

$$+\int_0^D \left(-\pi_{r+1} \cdot \frac{H}{2} + M' + \Theta' \cdot x'\right)$$

$$\left(-\frac{H}{2}\right) \cdot dx' + \int_0^D \left[-\pi_{r+1} \cdot \frac{H}{2} + M''\right.$$

$$\left. + \left(P_{r+1} + \Theta''\right)x'\right]\left(-\frac{H}{2}\right) dx' = 0$$

which gives, when the integrals are resolved :

$$-\pi_{r+1}\left(\frac{H.D}{2} + \frac{H^2}{12}\right) + \frac{H^2}{12}\pi_r -$$

$$-\frac{D}{2}\left(M' + M''\right) - \frac{D^2}{4}\left(P_{r+1} + \Theta' + \Theta''\right)$$

As :

$$M_{r+1} + M' + M'' - N.H = 0$$

and

$$N = \Sigma_1^r \pi + \pi_{r+1}$$

wet get : $M' + M'' = H\,\Sigma_1^r \pi + H\,\pi_{r+1} - M_{r+1}$ and also the shear :

$$T_r = P_{r+1} + \Theta' + \Theta''$$

$M_{r+1} = M_r + D.\,Tr$ and if we designate by M'^{r+1} the bending moment in the cross section in the middle of the panel r, $r+1$: $2Mr + DT_r = 2\,M_r^{r+1}$ which gives us finally (see for details Mr. Vierendeel's course).

$$\pi_{r+1} - \pi_r = -\frac{6D}{H}\Sigma_1^r \pi - \frac{6D}{H^2} M_r^{r+1}$$

a general formula for girders with parallel flanges having the same section and moment of inertia for both flanges and verticals.

Mr. VIERENDEEL then proceeds to investigate the limiting case where the I of the upper flange, becoming ever smaller and smaller, finally becomes infinitely small, the section of this flange always retaining a finite value; this amounts to assuming that the upper flange, hinged at the tops of the verticals, and acted upon by direct compressive load due to the action of π, the values q, Θ' and M' being nil. Then we shall have $\Delta x_{r+1} - \Delta x_r = 0$; let us remind the reader again that displacements due to direct loads are ignored.

The points of contraflexure are, in this border line case, located at the tops of the verticals and the theorem of MERTEN gives us :

$$\int_0^H (-\pi_{r+1} \cdot y)(-y)\, dy$$

$$+ \int_0^H (-\pi_r \cdot y) \cdot y\, dy$$

$$+ \int_0^D [-\pi_{r+1} \cdot H + M'$$

$$+ (P_{r+1} + \Theta'') x']$$

$$(- H) \cdot dx' = \Delta x_r \cdot \frac{d\pi_r}{d\pi_{r+1}}$$

$$+ \Delta x_{r+1} \cdot \frac{d\pi_{r+1}}{d\pi_{r+1}} = \Delta x_{r+1} - \Delta x_r = 0$$

In fact what we have is : $\Sigma \pi$ (independent of π_{r+1}) $+ \pi_r + \pi_{r+1} = 0$.

Developed the equations becomes :

$$- H^2.D.\pi_{r+1} + H.D.M''$$

$$+ \frac{H.D^2}{2}\left(P_{r+1} + \Theta''\right) - \frac{H^3}{2} \cdot \pi_{r+1}$$

$$+ \frac{H^3}{3} \cdot \pi_r = 0.$$

Now let us make similar substitutions to those effected in the first case, dealt with above, and we arrive with Mr. Vierendeel, at the general equation :

$$\pi_{r+1} = \pi_r + \frac{3D}{H} \cdot \Sigma_1^r \pi - \frac{3D}{H^2} \cdot M_r^{r+1}$$

Mr. Vierendeel shows that in order to calculate the values π we may assume the upper flange to be articulated at the tops of the verticals; the differences are practically negligible.

We will proceed on this basis of this remark in order to simplify considerably the work of calculating girders having non-parallel flanges.

Before taking up the investigation of this type of girder we will explain a particular contrivance for applying Merten's theorem allowing us to comprehend better how the general formula applying to the type of girder in question is arrived at.

Suppose any structure whatever acted on by forces (P, R) forming a group in equilibrium.

Now let us cause the systems of forces $(\overline{F}, -\overline{F})$ to act at any two points A and B on that object.

Nothing is changed thereby as regards the conditions of equilibrium of the structure and the reactions involved are independent of the values of $\overline{F}$.

We have :

$$\frac{dT}{dF} = \frac{1}{E}\int_o^s \frac{M}{I} \cdot \frac{dM}{dF} \cdot ds$$

M is a linear function of P, R and F.

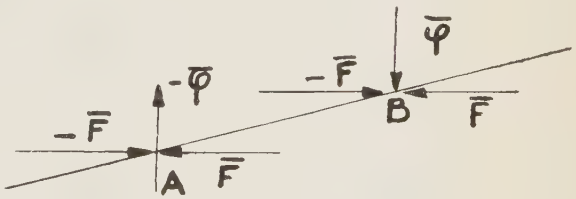


Fig. 5.

Now P and R are independent of F, hence the derivative $\frac{dM}{dF}$ is equal to the derivative of the moment resulting solely from the action of the forces F on the structure.

It is to be noted that we can replace the system $(\overline{F}, -\overline{F})$ at A and $(\overline{F}, -\overline{F})$ at B by one which is statically equivalent, i. e. the forces $\overline{F}$ and $\overline{\varphi}$ at B and $-\overline{F}$ and $-\overline{\varphi}$ at A, these being brought together in the relation :

$$F \cdot h = \varphi \times D.$$

Equilibrium remains undisturbed, the deflections and the stresses alone become modified.

Let us now discuss the second element in the theorem of Merten :

$$y_F + \Sigma y \cdot \frac{dP}{dF}$$

The equations of the statics provide us with the derivatives $\frac{dP}{dF}$; these equations are linear ones in relation to P, R and F, P and R being independent of F; the derivation affects only the forces which result from the static decomposition of the supplementary forces (F and φ.).

This method of considering the matter being applicable whatever the value of F, it is possible after developing it to make F = 0 which gives us a relationship corresponding solely to the action of the forces (P and R).

Now let us consider any panel whatever of a Vierendeel type girder, with straight under flange and polygonal upper flange, and let us act on the hypothesis of the upper flange having an infinitely small moment of inertia and therefore hinged at the top of the verticals.

The actual loading is set out below; let us add to that the supplementary one due to the forces F, — F, φ, — φ connected by the relationship $F \times h = \varphi \times D$.

The group of supplementary forces acts on the upper flange, incapable of being bent, resisting by compression alone, and, as we know, the displacements due to direct loads are not considered, and thus $\Delta x_{r+1} - \Delta x_r = 0$ which, in the second member of the theorem of Merten, causes the terms of the derivatives related to the forces F to vanish.

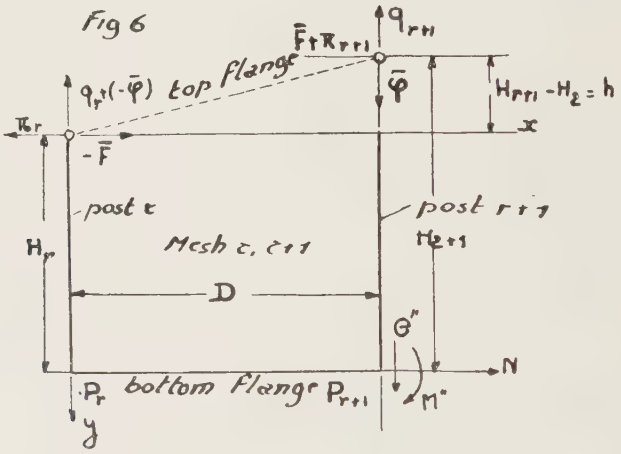


Fig. 6.

Taking into account the choice of axes of reference, Δy_r is nil, the second member of the theorem of Merten becomes reduced to

$$\Delta y_{r+1} \cdot \frac{d\varphi}{dF}.$$

The relation $F.h = \varphi.D$ gives :

$$\frac{d\varphi}{dF} = \frac{h}{D}$$

and Δy_{r+1} is the result of the rotation of the panel caused by the bending of the vertical r, a rotation equal to

$$\frac{\pi_r \cdot H_r^2}{2} \times \frac{1}{EI}$$

around the point of origin at radius D, or a displacement of

$$\frac{\pi_r \cdot H_r^2}{2} \times D \times \frac{1}{EI}$$

We have therefore :

$$\int_0^{H_{r+1}} (-\pi_{r+1} \cdot y - F \cdot y)(-y) \cdot dy$$

$$+ \int_0^{H_r} (-\pi_r \cdot y + F \cdot y) \cdot y \cdot dy$$

$$+ \int_0^{D} [-\pi_{r+1} \cdot H_{r+1} - F \cdot H_{r+1} + M''$$

$$+ (P_{r+1} + \Theta'' - q_{r+1}) x'$$

$$+ \varphi \cdot x'] (-H_{r+1} + \frac{h}{D} \cdot x') dx'$$

$$= \frac{\pi_r \cdot H_r^2}{2} \cdot D \cdot \frac{h}{D}$$

This developed equation leads us to the relationship (15) given by Professor Vierendeel, it being given that :

$$H_{r+1} - H_r = h \text{ and } F = \varphi = 0$$

$$\frac{\pi_r \cdot H_r^2}{6} (3 H_{r+1} - H_r)$$

$$- \frac{\pi_{r+1} \cdot H_{r+1}^2}{3} (3 D + H_{r+1})$$

$$+ M'' \cdot D \cdot H_{r+1}$$

$$+ (P_{r+1} + \Theta'' - q_{r+1}) \cdot \frac{D^2 \cdot H_{r+1}}{2}$$

$$= h \left[-\pi_{r+1} \cdot H_{r+1} \cdot \frac{D}{2} \right.$$

$$\left. + \frac{M'' \cdot D}{2} + (P_{r+1} + \Theta'' - q_{r+1}) \cdot \frac{D^2}{3} \right]$$

Evaluating M'' as a function of the values π and of M_{r+1} and also $P_{r+1} + \Theta'' - q_{r+1}$, as a function of the values of π and T_r, we finally obtain (for the developments of this, see Professor Vierendeel's course) :

$$\pi_{r+1} = \pi_r \cdot \frac{H_r^2}{2 H_{r+1}^3} (3 H_{r+1} - H_r)$$

$$+ \frac{D(3 H_r \cdot H_{r+1} + h^2)}{H_{r+1}^3} \cdot \sum_1^r \pi$$

$$\frac{3 D(H_r + H_{r+1})}{2 H_{r+1}^3} \cdot M_{r+1}$$

$$+ \frac{D^2(2 H_r + H_{r+1})}{2 H_{r+1}^3} \cdot T_r$$

the general formula for girders with vertical uprights, lower flange straight, and upper one polygonal (see Professor Vierendeel's course on stability of structures, Vol. IV. 1920 edition, formula No. 16, page 186).

There is no difficulty in arriving at the influence lines of the values of π when applying the general formula step by step and assuming a unit load, 1 000 kgr. for instance, in line with each vertical.

The reader will find in the volume of Mr. Vierendeel's course referred to above the calculation for q, Θ, N and M, after previously determining the value of π, a calculation which offers no difficulty.

Vertical posts.

The Vierendeel truss with a parabolic top chord when uniformly and completely loaded under dead weight for instance, acts as a parabolic arch rib with tie rod.

Bending moments and shear only occur in the vertical posts under the influence of rolling loads.

Therefore, we must for these posts make due allowance for the repetition and variation of stress and not lose sight of the question of impact and the important effect of sudden change of section upon the resistance.

All the bridges I know are built with curved and enlarged connections of the vertical posts to the chords.

Professor VIERENDEEL may then assume that the vertical deflections of both chords are practically equal.

It seems at first quite rational as the bending moments at these points are maxima.

But how are we going to calculate these curved connections?

For such beams with variable height, we very often apply what we call the Resal's method a and a' being the areas of the flanges α and α' their slope with regard to the longitudinal axis, h the distance betwen the center lines of the flanges, M and N the bending moment and the axial load, calling A_r a reduced area being equal to : $A_r = a.\cos\alpha + a'\cos\alpha'$, we get :

$$\text{average stress } f = \pm\frac{M}{h.a.\cos\alpha} + \frac{N}{A_r}$$

$$\text{average stress } f' = \mp\frac{M}{h.a.'\cos\alpha'} + \frac{N}{A_r}$$

This formula does not take into account the area of the web and its moment of inertia.

In several Vierendeel bridges in which the welding technique was identical with that employed in the case of the Hasselt bridge, the flanges of the gussets have been welded to the top edge of the lower chord.

Specially shaped steel plates, welded first to the top edge of the bottom chord connected this chord to the vertical post's flanges.

In this case :

$$\alpha = \alpha' = \frac{\pi}{2}, \cos\frac{\pi}{2} = 0,\ A_r = 0$$

according to Resal's formula.

Something else had to be found.

Professor VIERENDEEL took into account the stresses in the web and gave the formulas :

f average

$$= \pm\frac{Md}{I_w + d^2a\cos\alpha + d'^2a'\cos\alpha'} + \frac{N}{A_r}$$

f' average

$$= \mp\frac{Md'}{I_w + d^2.a.\cos\alpha + d'^2\,a'\cos\alpha'} + \frac{N}{A_r}$$

$A_r = a_w + a \cos \alpha + a' \cos \alpha'$

I_w = moment of inertia of the web,

a_w = area of the web,

d and d', distances of the neutral axis to the centre lines of the flanges.

The shear stress is given by :

$$\int_{d'}^{d} S.\, da = T - a f \sin \alpha - a' f' \sin \alpha''$$

But which is carrying at the joint with the lower chord the whole of the bending moment and the shear?

The web only; we have then to apply the simple and well known formulas :

$$f_{max} = \frac{N}{t \times h} + \frac{6M}{t \times h^2}$$

and

$$s_{max} = \frac{3}{2} \frac{T}{t \times h}$$

t = thickness of the web.

Now we understand better the reason of this wide gusset, but above the joint, what a dead zone of matter; of course, thereby we get some rigidity favourable to the assumption:

The vertical deflections of both chords are practically equal. It is all right in riveted work, is it still so in welded work?

There is still another loading rather hard to deal with for the designer, especially in case of riveted trusses. I mean the forces acting radially, normal to the curved flanges equal per unit of length to

$$\frac{a \times f \text{ aver.}}{R}$$

R being the radius of curvature, in this case the radius of the circle.

Transversally the flanges are even subjected to bending (see Bleich's theory).

This loading, often important, has to be carried by the rivets or the weld.

In case of riveted trusses, the strength of the joint as regards bearing and crushing will often be less than as regards shear because the plates are often thin compared with the diameter of the rivet. The addition of steel plates is often necessary, even in welded structures.

Many experiments including fatigue tests have been carried out, on different shapes of gussets (1).

Gussets with straight edges have been tested and the results have been satisfactory. Personally, I don't see any serious objection to the use of a gusset with straight edges, at least we ought to try it.

For a vertical post, if we don't take into account the web of the I beam, the law of variation for the section modulus is shown by the width of the post.

As the law for the bending moment is a straight line drawn by the point of contraflexure of the post if the bending increases, that is what happened in the Hasselt bridge after the initial failure, this straight line cuts the outline or contour in a' and a''.

The stress in the flanges is far from being uniform and shows a maximum at b, section overstressed well above the elastic limit of the metal as the calculations have shown in the case of the Hasselt bridge.

The length of the overstressed zone, adjacent to the welds is very short and this acts as a notch in the bar. The effect is one of abrupt change of section.

(1) See l'*Ossature Métallique*, March 1940, paper by Professor Campus.

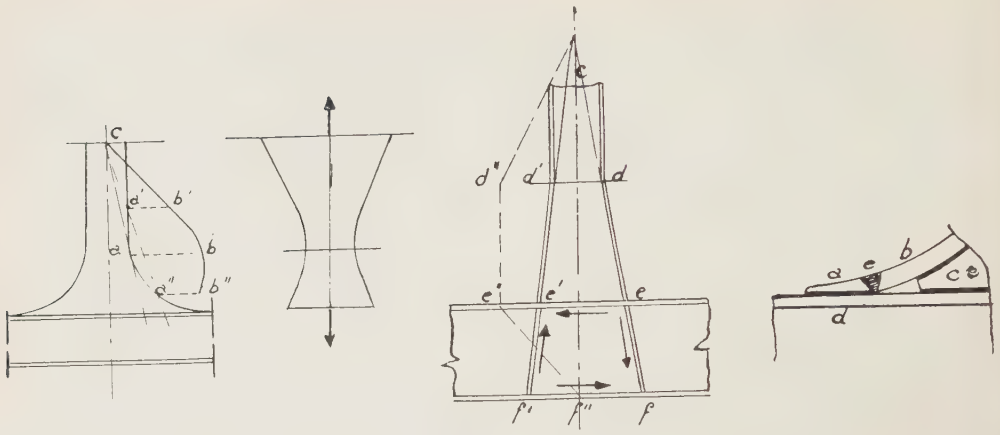


Fig. 7.

All the energy of the impact when the bridge fell in the canal was probably absorbed by the overstressed zones shown at r in the figure, the fractures occurred there without deformation.

As the law for the bending moment is a straight line, we ought to adopt a linear function for the section modulus variation.

Let us assume that the flanges carry the whole of the bending moment.

Section modulus

$$Z = 2a \left(\frac{h^2}{4}\right) \left(\frac{2}{h}\right) = a \times h$$

If the bending moment $M = m \times d$, d being the distance, of a section to the point of contraflexure $m = U'_{r-1} - U'_r$ (see theory of the Vierendeel truss explained above), we get : average stress f

$$\frac{M}{Z} = \frac{m \times d}{a \times h}$$

This stress ought to be kept constant all along the gusset plate, thus $\frac{d}{h} = \text{constant}$ and the height h of the gusset plate ought to vary in proportion to the distance of the section to the point of contraflexure.

Inconvenients of the solution. We get in d and d' a sharp change in the direction of the flanges and therefore *secondary* stresses will occur at these points, these

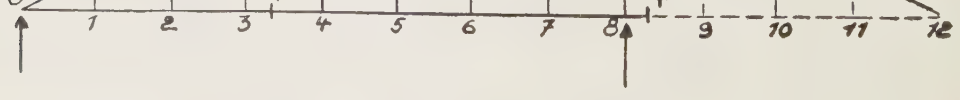


Fig. 8.

stresses may easily be lessened if we join the two slightly diverging flanges by a short transition curve.

In ordinary structural work, building frames for instance, horizontal stiffeners could be welded along $d\ d'$.

The welding of the stiffeners across a flange in tension is not the ideal solution, but this cannot be avoided if we wish to see the flow of stress in the post flanges to run straight in the chord web to be taken by shear as explained.

Designer : Professor DESPRETS.

Photo 4. — Vierendeel railway bridge at Malines.

At e and e', at the meeting points with the chord flange, we get unfavourable pointed or localised loads which have to be carried by stiffeners ef, $e'f'$ to the chord web and distributed by pure shear to the chord flanges.

And now we understand better, I expect, why this connection *post to chord* has always been a « stumbling block » for the designer unless he is able, like Professor Despretz does on riveted Railway bridges, to build chord and post with a *com-*

Designer : Professor DESPRETS.

Photo 5. — Vierendeel railway bridge at Herenthals.

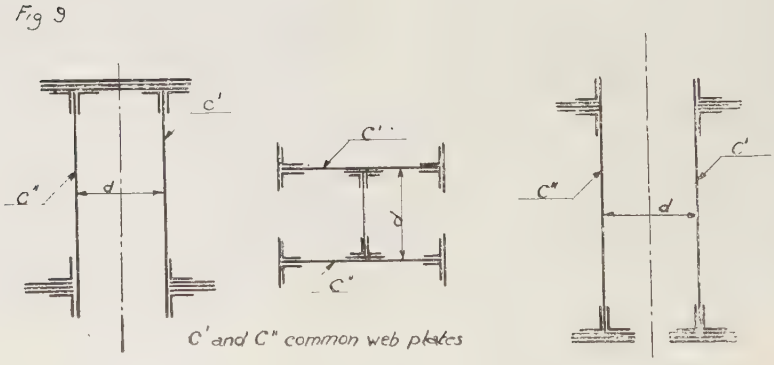


Fig. 9.

mon web plate, solution which much improves this difficult joint.

Let us remark that particularly in the case of highway bridges with heavy dead weight, it is quite rational to design the bottom chord as a tie of constant section throughout its length. I would like to draw attention to the necessity of rolling new profiles.

Welding requires already special ribbed

or nose flange plates unless we use split Grey beams or T sections (X weld, distortion of the flange, thick plates).

The angle bar, so useful for the connections in riveted work, is of no use as a connection in a welded structure as the fillet welds lie still further away from the centre of gravity of the section than the rows of rivets, the T section may be useful, it is not then a question of narrow flange limiting the size of the rivet, but the moments of inertia ought to be the same along the principal axis parallel to the flange and the web and so on.

The rolling mills in the near future will have to tackle this important problem of adequate profiles.

I read with great interest the paper of Mr. L. Reeve, published in the transactions of the Institute of Welding, about the Hasselt bridge.

Here are a few results of the examination of welded steel specimens, briefly :

Steel, basic Bessemer quality (fully rimming), chemical analyses normal, with the exception of a somewhat higher sulphur and phosphorus content in some specimens. Carbone 0.14, 0.15, 0.17, 0.20 %.

Mn 0.5 ... 1.02 % (not anormal if a medium high tensile steel is required);

Sulphur 00.6 ... 0.095 %; phosphorus 0.06 ... 0.084 % ; metallographic examination : oxide inclusions, marked sulphur and phosphorus segregation; mechanical tests : tensile test, ductility satisfactory, yield point round 15 t/in.^{2}, breaking stress : 29 t/in.^{2}, 30.2 t/in.^{2}, 27.1 t/in.^{2}

Elongation % in 4'' : 26, 30, 32, 33 %.

Izod tests definitely on the low side in the case of a heavy 55 mm. flange plate.

Steels very sensitive to the effects of strain ageing.

No exceptional hardness figures reported.

Serious defects in the macrostructure of several of the welds, cracks at the root of an important butt weld.

Complete absence of a sealing run at the back of this weld.

Lack of penetration at the root.

The failure must have been due to one or more of the following causes :

a) overstressing either generally or locally due to incorrect design.;

b) defective steel;

c) defective weld metal;

d) incorrect welding procedure and technique.

While the steel employed was unsatisfactory in certain respects, the whole blame for the collapse of the bridge cannot with any justification be placed upon the steel alone. The main cause of dissatisfaction is the comparatively low Izod value. The exact practical significance of low Izod value is difficult to state with any degree of certainty.

In particular, there is no definite evidence that such low Izods can be the direct cause of the type of crack known to have developed in this or similar welded bridges. It has been suggested that a steel with a low Izod value would propagate a crack more rapidly once such a crack had been started by other causes.

But even this requires confirmation and to support it in this particular case, it would have to be shown that the cracks

in the bridge were associated with the steels of low Izod values < 30 foot-pounds.

The special conditions required for producing strain ageing (250° 1/2 hour) were not present.

The microstructure of the steel was unsatisfactory in some respects and in certain cases (e. g. the coarse grain) may have been one of the causes of the unsatisfactory mechanical properties.

Sulphur and phosphorus contents were above normal specification limits.

Recent investigations indicate that the phosphorus content can be very much higher in *low carbon steel* than was formerly supposed without harmfully affecting the quality of steel; still, the content was too high for an important unit of a fusion welded structure. Quality of the weld metal is somewhat variable, it is improbable that actual mechanical failure could be ascribed to the weld quality.

There can be little doubt that the most unsatisfactory feature which has been exposed by the investigation is the defective nature of the welding procedure and technique.

The first point of importance is that in five Vierendeel bridges in which the welding technique was identical with that employed in the case of the Hasselt bridge, it has been demonstrated that concentrations of high internal stress occur in the region in line with the joint between the flanges of the gussets and the lower chord of the bridge.

This is the region a — b in the figure which shows where the initial fracture occurred in the Hasselt bridge.

In four of these bridges fissures have been discovered either in the welds or in the gusset flanges in this region and in certain cases distinct cracks have developed in the lower chord as indicated by the line c — d in the diagram as a result of the rupture of the gusset flange.

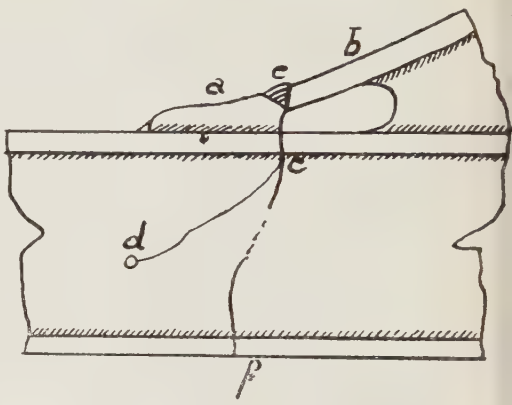


Fig. 10.

The examination of portions of the bridge recovered from the bed of the canal has led to the conclusions that the initial rupture occurred in a weld at e. The failure taking place abruptly is considered to have caused a sudden redistribution of the stress and brought about the failure of the lower chord as the result of shock.

Comparison of welding and riveting.

The superiority of welding over riveting in providing increased strength and resistance to leakage, (water or oil tanks), combined with saving in weight is generally recognised, but the *increased rigidity* obtained by welded connections *calls for great care* on the part of the designer who has to dispose his materials and connections so as to provide *as uniform a degree of flexibility as possible* throughout the

structure under the varying conditions which may be experienced in service and not the least difficulty in this respect is the avoidance of « *hard points* » where stress concentrations can occur.

For example, in riveted construction where angle sections are used as connecting mediums, local stress concentrations are relieved in many cases by the « spring » which can take place between the heel of the angle and the line of rivets.

Omitting the angles and replacing them by direct fillet welds would not provide a similar relief; it is therefore common to fit « *breathing* » pads in places where concentrated stress may occur and so provide for a better load distribution.

The greatest danger in this connection possibly lies where concentrated local alternating stresses occur as whilst it has been established that fatigue values for welded joints are very much better than when riveted, yet these values are low as compared with the ultimate tensile strength.

Extract of « The influence of gas cutting and welding upon ship construction », by Mr. H. W. Townshend.

Institute of welding.-Transactions. Jan. 1939.

Since the Hasselt bridge collapsed, great progress has been realized in welding technique. Thorough research work has been carried out in the Belgian laboratories and great experience has been gained.

I would like to summarize the results of these researches and experiments which have led to certain recommendations, rules and principles, admitted by every welding engineer when war broke out.

Electrodes. Choose a suitable, covered electrode in order that there shall be as little difference as possible between the parent metal and the weld metal.

When weld metal differs materially in composition from the parent steel, the bond or transition zone will be an alloy, the mechanical properties of which are sometimes surprising.

The mechanical properties of the parent metal and of the weld metal should approximate to each other.

Through the use of *covered* electrodes, it is possible to regulate the gain or loss of elements to get a sound transition zone where failure has sometimes occurred.

Nitrogen and oxygen can be absorbed into the weld metal when bare wire electrodes are used; it is then possible that the relative brittleness of bare wire weld metal renders it unable to provide adequate plastic deformation for the relief of local stress concentration.

Medium fluidity for the electrodes is advisable.

High fluidity makes welding in the vertical position difficult; overhead position ought to be avoided, if not prohibited.

Low fluidity is often the cause of lines of slag becoming trapped in the weld.

The electrodes should not require an excessive heat input; heavily coated electrodes cause higher shrinkage due to the increased heat input and the greater amount of penetration and of melted material.

The notch sensitivity as far as the main welds are concerned, can be avoided by machining and grinding the transition zone and the weld down, flush with the plate surface.

***

Weldability and ductility of the parent metal. Use a low carbon steel, such a mild steel of 38-44 kgr./mm², ultimate tensile strength 28 t./in² normalised ; yield point 15 t./in² (24 kgr./mm²).

Recent investigations, as we pointed out above, seem to indicate that the phosphorus content can be very much higher in *low carton steel* than was formerly supposed without harmfully affecting the quality of steel.

Beware of a steel with higher properties than those of the structural mild steel 28 t./in², sometimes required on the fact that the low working stresses permissible with steel 28 t./in² necessitate a high dead weight, restricting the bridge span.

Remember the difficulties encountered by the German State Railways with the welding of steel 52 (33 t./in², 52 kgr./mm²).

« Cracks appeared in the Zoo station single track railway bridge after it had been six months in service. These started at the fillet welded ribbed profile and ran up into the web.

» Shortly afterwards, cracks appeared during the erection of the Rüdersdorf Autobahn bridge after a drop in the atmospheric temperature (—10° C, 14° F)

These cracks occurred through the bottom flange of the girder and ran right up through the web. In both cases, the cracks showed no signs of deformation due to a state of triaxial stress and to excessive hardening. »

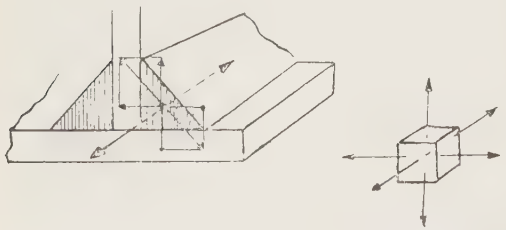


Fig. 11. — Shrinkage stresses in a common side fillet.

Constituents St. 52.

C 0.17 %, Si 0.45, Mn 0.87, Ph 0.037, S 0.017, Cu 0.49, Cr 0.17.

Yield point 22.4 t./in², ultimate tensile strength 33 t./in².

Don't forget that increased thickness of sections, of nose plates for instance, makes it difficult to maintain the specified yield point and the uniform mechanical properties throughout the whole section. The Belgian Railways took as limit 40 mm. let us say less than 2″ for steel 38.

Remember that the tendency to hardening in steels is proportional to its carbon content, that the rate of heat conduction is proportional to the thickness of the parts welded.

The cooling may be so rapid that hardening will occur in the immediate neighbourhood of the welds.

What about *ductility* ?

I read in the Welding engineer's pocket book :

« The real necessity for the property of ductility is to permit the several parts of a structure to withstand local stresses.

» The location of these stresses may be assumed, but they cannot be calculated and it is not possible to anticipate them in full.

» We know such stresses are extremely local in their incidence and the volumes of metal subjected thereto are microscopic, thus if the material does not permit of

plastic deformation, a minute fracture will occur, ultimately spreading to serious proportions.

» For this reason, the ductility of each and every minute portion of the structural elements is of serious import. »

We all know that unusually high peak stresses originate in the weld and the immediate adjacent zone of the parent metal in consequence of hindered deformation.

Let us assume a pair of plates quite free to move and quite unrestrained, which are to be buttwelded along a veetrough. The plates will expand, but the metal near the weld will be restrained because of the large mass of metal on each side of the weld. A narrow zone, on each side of the weld, will undergo plastic flow; the yield point of steel is quite low at the high temperatures (9 tons/in^2 at 1 200° F instead of 18 tons/in^2 at room temperature).

As the plates cool to room temperature, the weld metal and the base metal which were subjected to plastic flow during heating, will tend to contract to a length less than the original, but will be restrained by the base metal outside the plastic flow zone which tends to contract only to its original length.

In the case of freely movable pieces, compression stresses corresponding to the tensile stresses in the weld occur as reaction stresses in the zones further away from the weld, remaining in equilibrium with one another $F_3 = F_4$.

Transversally, residual stresses will also be locked up so that $F_1 = F_2$ and the equations of $F_1 a_1 = F_3 \cdot a_2$ statics will be satisfied.

The magnitude and the direction of the internal stresses induced may be obtained by measuring the distance between gauge points *after* welding and then again after they had been cut up into small pieces to relieve the stresses induced by welding.

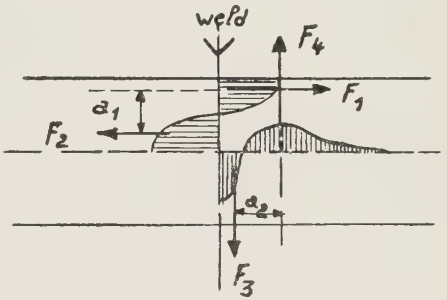


Fig. 12.

Many experiments have been carried out and the stresses found show that the centre section of the plates is subjected to tensile stresses sometimes as high as the yield point in two perpendicular directions.

And still this is quite a simple, elementary case.

Bi-axial and tri-axial stress conditions occur in most welded joints and it is possible that failure may occur without measurable deformation as in the case of fatigue failure.

We attribute much importance to ductility; it is indeed a fact that under uni-axial stress conditions high ductility allows a high degree of deformation when material is overstressed.

The structural stresses in contrast to the weld stresses are not of a spatial nature, there are no abrupt reversals of stress and they don't raise the yield point.

The normal internal elastic welding

stresses do not however exert any seriously harmful effect on fatigue resistance.

It must be assumed that these stresses are decreased in the first stages of fatigue loading without leading to the formation of cracks.

The influence of elastic shrinkage stress which can assume, I admit, high values, must therefore not be overestimated.

If these act in conjunction with applied loads, the stresses in a structure would often be so high that its safety could be assumed to be impaired.

MATHAR gave a method of ascertaining the stresses in a plate for instance, by drilling a hole, recording and measuring the elastic deformation by means of a precision extensometer or by a mirror during drilling.

Shrinkage stresses in welded joints are of the same nature as those which arise in rolled broad flange girders at the point of junction of the flange and the web.

The internal stresses are decreased immediately the structural part is loaded, the elastic stresses causing a very limited plastic deformation which decreases the internal shrinkage stresses.

The same applies in those parts of welded joints where peak stresses exceed the yield point and which, through plastic deformation under an applied load, cause an evening of the stress distribution.

This evening of stress, will be achieved without cracks if our weld and base metals are ductile, we could not emphasize too much this requirement : *ductility*.

***

Various welding techniques can be used to decrease undesirable high welding stresses in the joints; among the more important suggestions for the designer, let us point out that the cross section of the weld should be kept as small as possible, all the welds being the result of calculation and the number of fillets in one zone being reduced to a minimum.

Reduce the number of stiffeners and don't weld these stiffeners to the flange in tension; use fitting plates between the stiffeners' ends and the flange in tension to ensure the continuity.

These fitting plates must be thick enough (7/8″).

To prevent the buckling of the flange in compression of a plate girder, short stiffeners are sometimes welded only to the part in compression of the web and distributed judiciously with the main stiffeners against buckling of the web plate.

If an additional plate has to be welded to a flange, its ends ought to be machined in order to get a wedge shape, the thinning of the ends in height reducing the notch sensitivity.

It is not necessary to reduce at the same time the width of the plate, the flow of the stresses is too much disturbed and cracks may appear.

Butt welding in plates along a 45° slope is rather difficult, we have given up doing so.

Fabrication facilities. I read these few remarks about welding in shipbuilding, they apply just as well to bridge work.

« Due regard has to be made of the methods to be employed in executing the

work, this may have an important bearing on the design.

» Generally the governing factor in this connection is the maximum weight which can be handled in one lift so as to preconstruct any part of the structure in as large a unit as possible prior to transporting and erecting on the building site. »

Sometimes the welding of the cross girders and stringers have been carried out in the workshops.

This procedure has not always been entirely satisfactory. Briefly, the fabrication facilities should be such that the component parts which are to form the preconstructed unit can be quickly and easily assembled and kept in position, also movements which are likely to take place whilst the welding operations are proceeding can be directed and kept under control.

It is also an advantage to have uplifting and for high plate girders rotating appliances so that each unit can be altered to give the best position for easy welding.

In any case, there must be a close and accurate fit between the pieces to be welded so as to avoid oversized welds.

Don't forget that shrinkage stresses are not always harmful.

We very often can arrange that these tension stresses due to shrinkage will counterbalance or neutralize some compression stresses due to loading.

Let us show how the sequence of welding may affect the sign of the stresses in the welding of two ordinary rolled joists of the same section.

Arcos, a leading firm in welding in Belgium and abroad has used, after careful tests in laboratory, shrinkage stresses to reduce stresses in a bottom chord joint of a Vierendeel bridge.

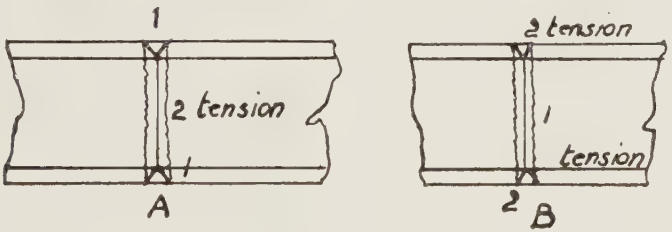


Fig. 13.

Double gussets with parabolic ends were welded on each side of the web in the order shown. We know that an uniformly loaded cable takes up a parabolic shape. We may expect that the tension stresses due to shrinkage will pull the gussets uniformly and therefore induce compression stresses in the web of the bottom chord relieving the vertical joint subjected to pull.

A relief of 6.5 kgr./mm^2 or 4.1 t./in^2 was measured with Huggenberger's extensometer in the vertical joint between the chord and a post.

The designer will have thus to make

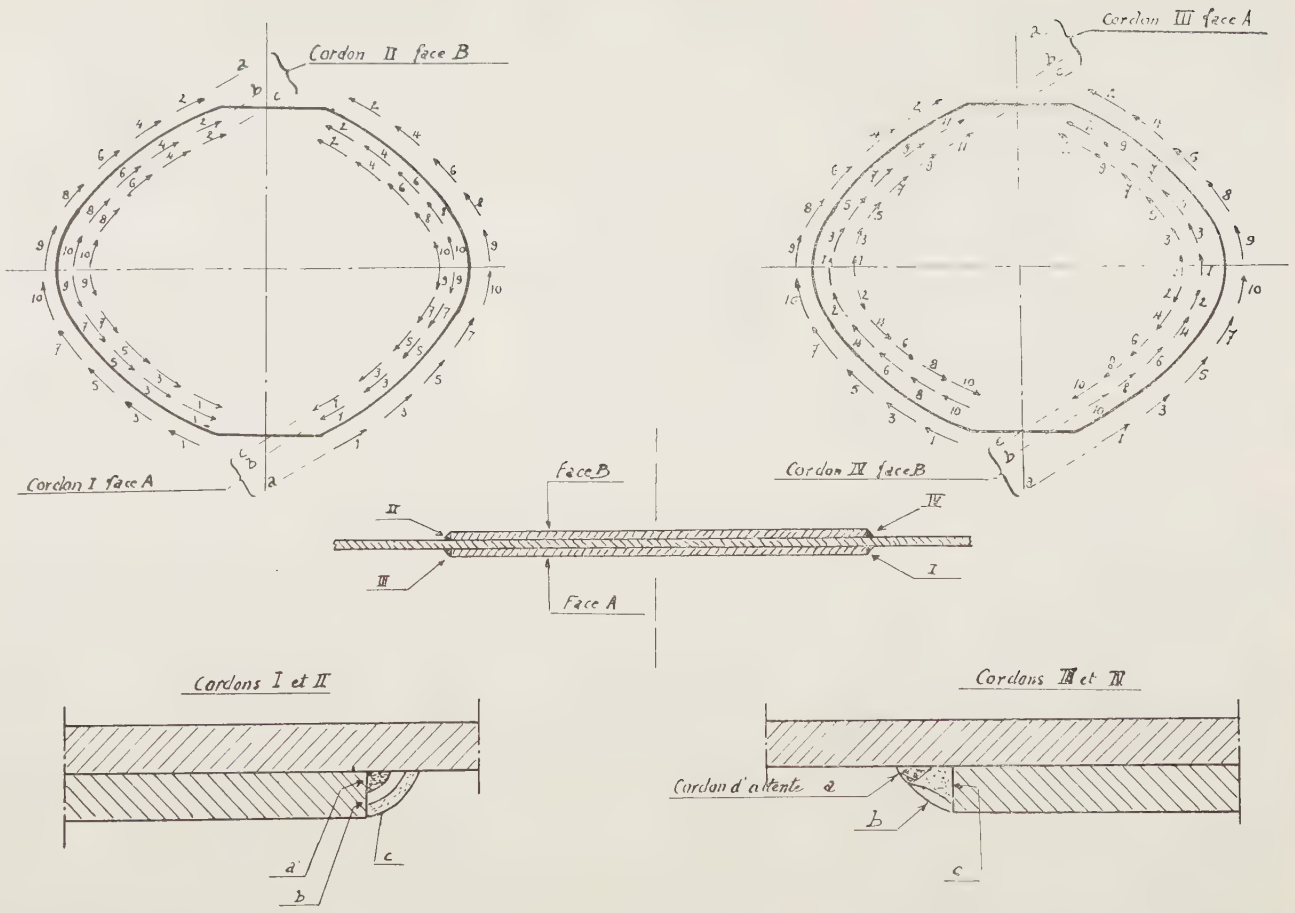


Fig. 13*bis*. — Order of welding the cover plates.

Cordon = fillet weld. — Cordon d'attente = starting fillet weld.

a careful study of the distribution of the welds and of the choice of the welding procedure to discuss in complete agreement with the workshops and the men in charge of the erecting on the building site.

To follow a general rule in the workhops, weld as many joints as possible in an unrestrained state of the members; it is desirable that the direction of welding should be away from the point of restraint and towards the point of maximum freedom.

In other words, weld away from a welded seam at right angles to the welding.

Clamping will reduce warping, but will induce internal stresses.

Use skip welding, step back welding or skip step back welding to prevent distortion and to reduce locked up stresses.

Their principle is to keep the expanding zones sufficiently narrow and sufficiently close to the contracting zones so that they tend to stress, relieve or neutralize each other.

Peening is an effective method of reducing stresses and partly correcting distortion or warping. We require *sometimes* for special welds, *always* for repairing, the preheating of the pieces to relieve the stresses.

Annealing is of course very difficult in bridge work, but we can deposit an additional run whose heating effect is favourable for the underlayers and afterwards grind this run away. We said that each job must be studied to determine the best procedure to follow; we require very often back to back welding, welders working in conjunction along the axes of symmetry of a joint.

Don't neglect the cleaning and hammering of the different layers, the steadiness of the electric current (amp. and volts).

We advise the use of small electrodes with low welding currents in order to get less distortion and lower residual stress.

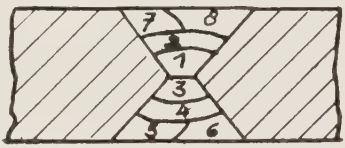


Fig. 14.

Now a few words about the common faults in welds and how to detect them.

One of the worst faults is the lack of fusion of V or X welds.

We require the back welding on the root, the sealing after cutting out and refreshing the root.

Undercutting is to be avoided.

These two faults increase the notch sensitivity and impair the fatigue strength of the structure or the endurance limit.

So does all discontinuity due to undersize, oversize or uneven weld.

Require a close and accurate fitting to avoid a too great volume of weld, a clean and neat machining of the edges to avoid microscopic cracks.

Sometimes slag becomes trapped in the weld bead and may lessen the cohesive section of the metal and concentrate the stress on the neighbouring parts.

Much could be said about these inclusions of fluxing agents, about constituents trapped in the weld metal and developing gas pressures and blowholes.

The metallurgy of steel welds, their

physical and chemical conditions are interesting fields for investigation.

Account has to be taken of the extremely high temperature reached before casting and the extremely short duration of the liquid state of the weld cast in bond with the parent metal as compared with an ordinary ingot.

Crater cracking, a rather common occurrence, may lead to weak points although often the following bead entirely heals the crack, one of the advantages of the multilayer welding.

For the butt welding of plates, we require the use of small transverse plates tack welded to the main ones, forming a trough; the bead is welded too long and the extremities with craters are taken and ground away after welding.

In other cases, we do away with the craters by working in pairs.

As the one welder is about to finish his electrode, the other welder takes the arc from him, the welding goes on continuously (automatic welding).

But we see by all these precautions how the human factor intervenes and may influence the final result.

Among other qualifications required of the welding personnel, scrupulous care plays a great part.

A simple visual inspection of welds is not sufficient to judge the quality of a weld.

Ways and means have been sought to control the finished product.

Let us mention first the boring of holes with a specially devised drill and the etching, carried out with an appropriate chemical reagent. As regards non destructive testing, I would like to say a few words on the magnetic and on the X ray methods.

In some cases γ rays have been used.

The magnetic method of inspection gives very interesting results on machined specimens, but does not record essential faults in unmachined specimens.

This method is used to test shafts for instance.

The X ray method is at present the only one which immediately detects a faulty weld or the use of unsatisfactory electrodes (inclusions, blowholes).

The innovation of the shockproof and rayproof shield has made this method a practical proposition. The interpretation of the radiographs (has the weld to be rejected or not?) is sometimes very delicate and it requires some experience to draw conclusions.

Calibration wires of different diameters are used to appreciate the importance of the cracks.

Let us remember that the weld is placed between the film and the X ray tube.

The radiation penetrates the weld and impinges on the film.

Take the case of a vee weld with lack of fusion at the root.

This lack of fusion corresponds to a reduction in thickness of the assembled plates, the radiation striking the film will not be uniform, being stronger along the root of the vee which will be defined as a thin black line on the film.

Insurance regulations and the technical specifications of the Belgian Railways specify that the main welds shall be radiographed by X rays.

Most firms have now their own X ray

plant installed in their workshops, plant which has much improved the standard of work and allows research work by the structural engineer and foremen in charge of the welding technique.

We have applied the X ray control on a large scale in Belgium, but I would like to say before finishing, a few words about the vibration test on welded bridges.

To this effect, we use a swinging tester in principle composed of two eccentrically rotating bodies, these bodies rotating in opposite direction.

The centrifugal force F at any moment for each body may be resolved into its vertical and horizontal components. The horizontal components balance each other and the vertical components act upon the bridge according to a sinusoïdal law between two limiting values — 2F, + 2F.

Fig. 15

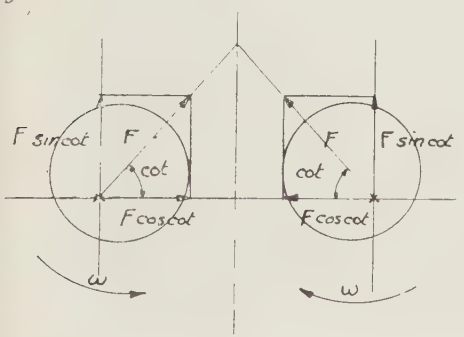


Fig. 15.

The speed of rotation is gradually increased, the deflection of the bridge is recorded till resonance occurs.

We have attended different tests carried out by Professor Rôs of Zürich on plate girders, their own vibration characteristics were found to be 6 Hertz (6 vibrations per second) if we remember. (Bridge of Gellick on the Albert Canal.)

Tests are made, let us say one year after to see if this characteristic 6 Hertz is maintained.

If meanwhile some failure has occurred in some welds, the vibration frequency decreases.

The damping arises from lack of elasticity.

This test has been carried out on a large scale by the Swiss Railways.

I have attempted to give a general outline on the stress analysis, the design and the welding of Vierendeel girders.

I have also tried to describe the fresh fields for research which were open for investigation when war broke out.

I insist on the connection of the vertical posts with both chords of a Vierendeel girder.

The same connection occurs in multi-storied buildings between the columns and the girders, but here the impact problem is far from being so important.

I feel sure that in the future, the research workers in Belgium and abroad will carry on, collaborating more closely to solve all the difficulties encountered in this new technique of welding applied to structural work.

Fresh applications of welding will be found and the existing ones will be extended.

I again draw the attention to the necessity of the rolling mills tackling without delay the important problem of adequate profiles.

Isolated cases of failure may lead sometimes to a temporary slackening of

progress, but we are quite confident that the difficulties will always be overcome.

The thorough research work carried out by the laboratories, together with the experience gained from the number of bridges already in service gives us every hope in the future of this new process in structural work.

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Franki-Bagon concrete railway sleepers,

by J. DELVAUX,

Ingénieur A. I. Lg.

(La Technique des Travaux.)

Technicians have for a long time been endeavouring to replace the wood sleeper by concrete or metal sleepers.

Certain systems have successfully stood up in practice but none of them have taken the place of the wood sleeper which, up to the present, is still preferred on nearly all the railways of the world.

However, the wood sleeper is not always the perfect solution. In tropical countries, for example, it is destroyed by termites. In our countries, the shortage of wood has created a very grave problem for the railways, since the renewal of the permanent way involves the use of several million sleepers a year. Before the war, 5 million a year were used in France and England, 3 million in Italy and about 800 000 in Belgium.

Fig. 1. — Franki-Bagon sleepers on the Brussels-Ghent line at Ternat.

The full effects of the fact that maintenance operations could not be carried out during the war on the European railways are now being felt, and the number of sleepers to be replaced has reached a truly impressive total.

The small amount of timber available

is immediately distributed to the different branches of the industry, and there is no hope of sufficient supplies being available for several years to make good the deficit.

Other materials must therefore be considered.

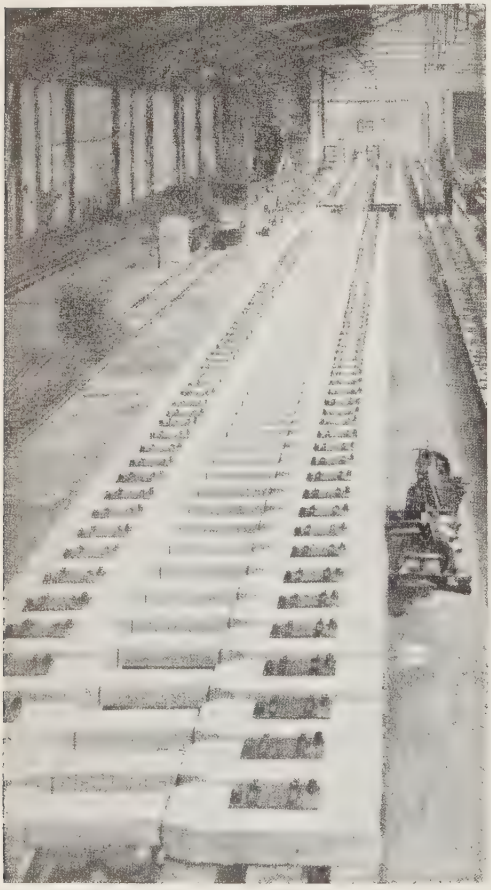

Fig. 2. — Plant manufacturing Franki-Bagon sleepers. View of the manufacturing shop.

Before the war, certain kinds of concrete sleepers had aroused considerable interest in spite of the abundance of wood sleepers at the time.

At the present time, when new techniques have been developed, the question of concrete sleepers is again to the fore, since thanks to these techniques their mass production is now possible.

In addition, war time experience has proved that synthetic materials are more satisfactory in meeting certain requirements than the materials habitually employed to date.

There is nothing to prevent this being the case with railway sleepers too.

Characteristics of a wood railway sleeper.

The problem to be solved is however rather complicated.

The dynamic loading of the sleeper, the effect of the joint, the variable and often indeterminate mechanical value of the ballast, the results of a thaw, the influence of tamping and the removal of the ballast are so many factors which it is difficult to define exactly, and this lack of precision makes it impossible in practice to make any strictly organic calculations for a sleeper.

However it is possible from actual experience to define the characteristics of a satisfactory wood sleeper, as proved by the classical investigations into its mechanical strength, flexibility and endurance.

A satisfactory sleeper should in addition :

1. Maintain the correct gauge of the track;

2. Prevent creep of the track as a whole owing to the way it is bound in with the ballast;

3. Be easily transported and put into place;

4. Not involve any alteration in the current practice as regards laying the track, maintaining it, and the equipment involved.

A wood sleeper is a rectangular unit of 2.60 × 0.26 × 0.16 m. (8′ 6 3/8″ × 10 1/4″ × 6 5/16″).

Oak, which is generally used, has a modulus of elasticity of 100 000 kgr./cm² (1 422 330 lbs. per sq. inch.), its resistance

to traction in the direction of the grain is 700 kgr./cm² (9 956 lbs. per sq. inch.) and its resistance to crushing in the perpendicular to the grain is about 400 kgr./cm² (5 689 lbs. per sq. inch).

When laid on the ballast and supported by the two areas of compression of which the rails form the axis, it undergoes deflections, either simultaneously or alternately from one or other rail, the deformation of the ballast under heavy passing loads reaching according to the specialists differences of some 7 mm. (9/32″).

Characteristics of a concrete railway sleeper.

To meet these severe conditions satisfactorily, it is not possible to make use of a material — steel or concrete — whose characteristics are very different from wood unless preliminary precautions are taken.

The general profile of the part will obviously differ according to the material used; the general outline of a metal sleeper is completely different from that of a wood sleeper.

The same applies in the case of concrete, the characteristics of which are as follows :

Coefficient of elasticity in the case of vibrated concrete	400 000 kgr./cm²	(5 689 330 lbs. per sq. inch.)
Limit of breaking due to crushing . .	600 kgr. cm²	(8 533 lbs. per sq. inch.)
Limit of breaking due to extension .	30 kgr./cm²	(426.7 lbs. per sq. inch.)

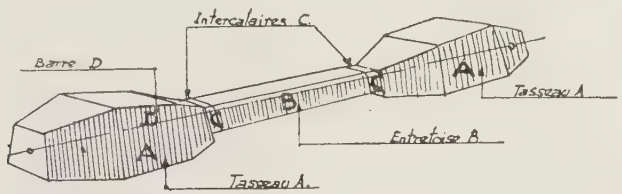

Fig. 3. — Diagram showing the arrangement of a Franki-Bagon sleeper.
A. = Block. — B. = Tie beam. — C. = Packing pieces. — D. = Rod.

It is impossible to make a sleeper in reinforced concrete as a continuous beam having the same resistance and flexibility as a wood sleeper.

Prestressed concrete makes it possible to approach these two characteristics and it is thanks to this remarkable process that a monolithic concrete sleeper has been able to stand up in practice.

Many attempts have been made to manufacture concrete sleepers consisting of elements linked together by mechanical articulation. Such sleepers got out of shape according to the changes in the ballast and did not allow the track to return to its initial position by itself.

The Franki-Bagon concrete sleeper.

Since 1942, the « Pieux Franki » Company has investigated and perfected a new kind of concrete sleeper, invented by the engineer Arnold Bagon.

Its mechanical resistance is comparable with that of a wood sleeper. Its endurance as measured in very severe laboratory tests appears to be superior to that of a wood sleeper. For more than a year it has stood up in practice to trials on a line with fast traffic.

Characteristics and manufacturing process.

This sleeper consists of two blocks and a tie beam of reinforced concrete between which packing pieces are inserted (fig. 3). These pieces are made of « resarm » whose modulus of elasticity is less than that of the concrete. Its characteristics are as follows :

Resistance to compression	1 500 kgr./cm²	(21 335 lbs. per sq. inch.)
Modulus of elasticity, approx.	30 000 kgr. cm²	(426 699 lbs. per sq. inch.)
Water absorption	0.1 %.	

The five parts (blocks, tie beam and packing pieces) are assembled by a special steel rod which is strained and fastened down at the ends.

The association of the strained rod and the packing pieces between the blocks and the tie beam results in a stressed joint which gives the sleeper its chief characteristics.

The blocks and tie beams are manufactured by vibration combined with a light compression. This process makes it possible to obtain standard concrete parts whose resistance to crushing is more than 650 kgr./cm² (9 245 lbs. per sq. inch.) after 28 days.

The assembly rod, in special steel, undergoes heat treatment which increases at 100/110 kgr./mm² (63.49/69.84 Engl. t. per sq. inch.), its apparent elastic limit at 0.2 %.

The rod is protected by two coats of bitumin and by being lapped in bituminized canvas.

The blocks and tie beams, which are taken out of the moulds immediately after vibration is completed, are finished off by a patented process which makes it possible to obtain the correct gauge for the rails with a tolerance of 1 mm. (3/64″).

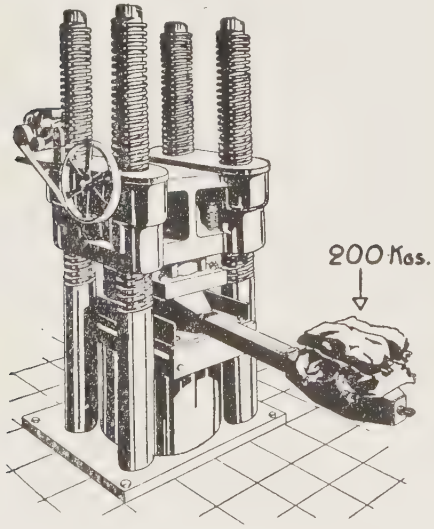

Fig. 4. — Testing a Franki-Bagon sleeper. Static test.

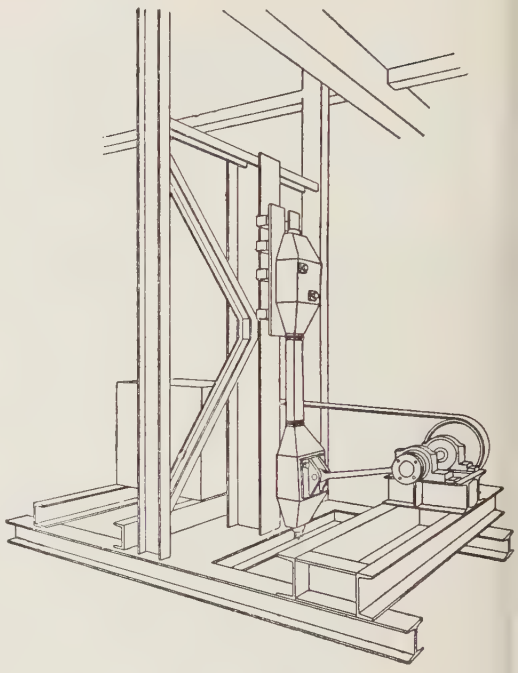

Fig. 5. — Testing a Franki-Bagon sleeper : alternating stress test, by means of a machine installed by the Pieux Franki Company in the Laboratories of the University of Liège.

Experiments carried out in the test laboratories of the Civil Engineering Building School at the University of Liège gave the following results :

1) The Franki-Bagon sleeper when at rest behaves like a single piece sleeper. It has perfect stability of shape and can be handled without any apparent bending. If the sleeper is fastened at one end and submitted to a load of 200 kgr. (440 lbs.) at the other, it bends less than 4 mm. (5/32″) at the end, as fig. 4 shows;

2) In service, the sleeper has all the necessary flexibility to meet traffic require-

ments. It can stand up to alternating stresses for a practically unlimited number of times.

When fastened at one end and connected at the other to an excentric mechanism as shown in fig. 5, the sleeper stood up to

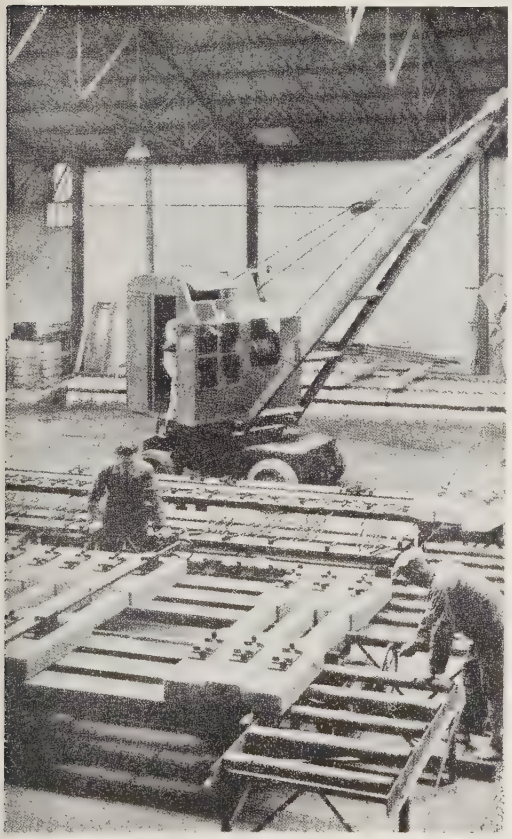

Fig. 6. — Storing Franki-Bagon sleepers.

25 million alternating stresses of 15 mm. (19/32″) at a frequency of 290 per minute without any apparent signs of fatigue.

This test, which takes two months at least to carry out, corresponds to twice the number of stresses an oak sleeper will undergo during its whole life.

The Franki-Bagon sleeper combines two apparently contradictory qualities : it is stable in the static state and deformable in service, its deformability being perfectly elastic.

Consequently, it combines with the strength of concrete the essential flexibility which only the wood sleeper could give up to the present.

Types of rail fastenings.

The sleepers supplied to the Belgian National Railways are fitted with « Geo » bearing plates fastened to the concrete by square-headed bolts (fig. 8).

This type of fastening adopted by the Belgian National Railways has various advantages : in particular it prevents creep. Tests made on a section run over by trains at high speed show that these sleepers with « Geo » bearing plates behave very satisfactorily.

Other types of fastenings can also be used, for example clips and hammer-head bolts.

Manufacture.

The Franki-Bagon sleeper lends itself to mass production. A manufacturing plant has been set up near Brussels (fig. 2, 6 and 7).

Tests.

Sleepers were laid at the end of 1945 at Ternat on the Brussels-Ghent line, over which express trains run at a speed of 120 km. (74 miles)/h. (fig. 1).

The results were very satisfactory as regards resistance of the sleepers, maintenance of the gauge, behaviour of the ballast, maintenance, etc.

Advantages.

The Franki-Bagon sleepers have the following advantages :

1) *Flexibility.* — Comparable with a

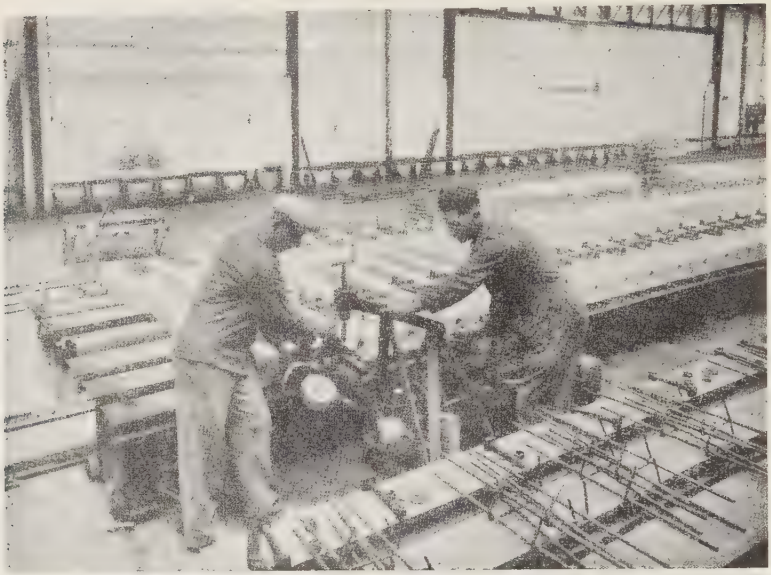

Fig. 7. — Plant manufacturing Franki-Bagon concrete sleepers near Brussels. Putting the rod under tension.

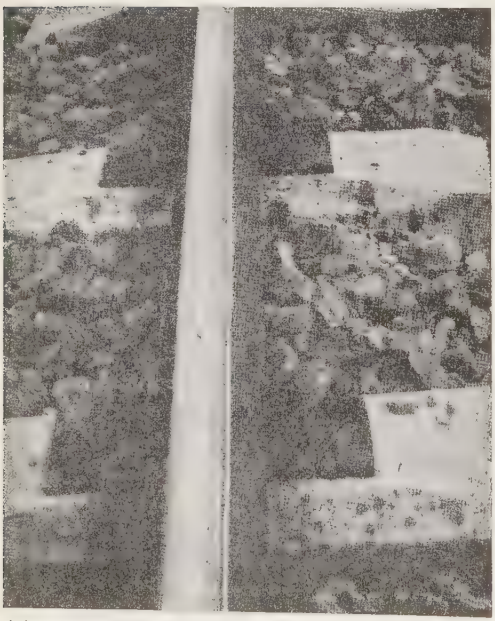

Fig. 8. — « Geo » bearing plate. This plate is fastened to the concrete by square-headed bolts. It has various advantages, such as preventing rail creep.

wood sleeper. This flexibility is an advantage, not only from the point of view of the strength of the sleeper, but also as regards the rolling stock.

2) *Ease of taking apart.* — By means of a small portable jack, it is possible to check the tension of the rod and correct it if needs be.

It is also possible to take up the sleeper and replace a damaged part without removing the rail. The replacement of a rod or tie beam only takes a few minutes.

3) *Re-use.* — If a derailment occurs, wood sleepers are only slightly damaged. On the other hand, concrete sleepers suffer much damage.

In the case of the Franki-Bagon sleepers, the damage is limited as it is possible to make up new sleepers from any undamaged portions, and this operation can be carried out on site.

4) *Manufacture.* — The sleeper lends itself to mass production with relatively simple equipment.

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P. GHILAIN,

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CHAPPÉE. — Influence sur les **réseaux de transport et de distribution d'énergie de la traction** à courant monophasé 50 périodes. (5 000 mots & fig.)

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Fresh fish traffic. Western Region **insulated vans.** (300 words & fig.)

621 .338.(485)
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Light electric railcars. Swedish 15 000-Volt type. (600 words & fig.)

385 (071 (42) & 385 .586 (42)
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Mobile railway schools. (500 words & fig.)

625 .28 (42)
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621 .431.72 (62)
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Fairbanks-Morse to build consolidation line of Diesel road locomotives. (1 000 words & table.)

1948 **385** (072 (73)
Railway Age, October 9, p. 58.
Plans unveiled for A. A. R. laboratory. (1 000 words & fig.)

1948 **625** .28 (73)
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1948 **656** .254 (73)
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Improved 65- and 80- ton General Electric switchers. (300 words & fig.)

1948 **656** .212 .5 (73)
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Builds second large **retarder yard to improve operation.** (4 000 words & fig.)

1948 **625** .234 (725)
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Mexican Government installs **radiant heat** in the private car of President Aleman. (4 00 words & fig.)

1948 **625** .174
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RISK (J. W.). — **Know-how and equipment will k Canadian National open during coming winter.** (words & fig.)

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1948 **656** .224
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NOTARI (M.). — Le **vibrazioni elastiche** quasi-armo- niche **nel biellismo dei locomotori elettrici.** (2 000 parole fig.)

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DERENS (L.). — **Moderne locomotiefconstructies** en haar antecedenten. (15 000 woorden & fig.)

1949 **625** .42 (460)
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De ondergrondse in Madrid. (800 woorden & fig.)

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VAN DER MEULEN (J. H.). — **Exploitatie van grensbaanvakken.** (2 500 woorden.)

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16. 385. (02]

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SAMUELSON (S.). — Scheme for an **undergroun railway** between Toronto and Montreal. (1 600 wor & fig.)

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La **coordination rail-route** en Suisse. (3 000 mots.)

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Bulletin de documentation S. C. E. T. A. : avril, p. 1; mai, p. 1.

Organisation des transports terrestres en Grande-Bretagne. (7 000 mots.)

Bulletin technique de la Suisse romande (Lausanne.)

1949 **691** (493) & 721.9 (

Bull. techn. de la Suisse romande, 26 mars, p. 77.

MAGNEL (G.). — Les **applications du béton contraint** en Belgique. (2 500 mots & fig.)

1949 **656** .254 (

Bull. techn. de la Suisse romande, 16 juillet, p.

BAUMGARTNER (J.-P.). — Le problème « **dispatching** » sur les Chemins de fer suisses. (3 mots.)

1949 **621** .31 (

Bull. techn. de la Suisse romande, 16 juillet, p.

La **production et la consommation de l'énergie trique** en Suisse pendant l'année hydrographique 1 1948. (3 000 mots, tableaux & fig.).

Bulletin de l'Union internationale des Chemins de fer. (Paris.)

1949 **656** .235

Bulletin de l'Union intern. des ch. de fer, mars, p.

MERMET. — La nouvelle **tarification des marc dises** de la S. N. C. F. (Tarifs de réglementation). (6 mots.)

)49 **016** .385 (44)
etin de l'Union intern. des ch. de fer, mars, p. 98.
RÉCHOT (J.). — Le **Centre de Documentation** nique de la S. N. C. F. (2 000 mots.)

)49 **656** .211.7 (481 + 489)
etin de l'Union intern. des ch. de fer, mars, p. 100.
ÖKEN (Kr.). — **Service de ferry-boat** entre la vège et le Danemark. (1 200 mots & cartes.)

)49 **621** .33 (44)
etin de l'Union intern. des ch. de fer, mars, p. 108.
a **table électrique de calcul** de la D. E. T. E. (Division Etudes de Traction électrique) de la S. N. C. F. 00 mots & fig.)

L'Industrie
Voies ferrées et des Transports automobiles. (Paris.)

)49 **621** .431.72
ıdustrie des Voies fer. et des Transp. automobiles, mars, p. 399.
ERRIN. — Note sur la **transformation de locomotives apeur en locotracteurs Diesel.** (600 mots & fig.).

)49 **385** (09 (69)
ıdustrie des Voies fer. et des Transp. automobiles, mai, p. 420.
ARTILLEUX. — Les **Chemins de fer de Mada-car.** (3 000 mots & fig.).

Notre Métier. (Paris.)

)49 **621** .138.1 (44)
re Métier, 18 avril, p. 8.
ORCE (G.). — Le nouveau **dépôt de Chaumont.** 00 mots & fig.)

)49 **621** .138.5 (44)
tre Métier, 16 mai, p. 6.
OISSONNIER (J.). — **Méthode moderne de répa-on accélérée des locomotives.** (1 000 mots & fig.)

)49 **385** (09 (61 + 64 + 65)
re Métier, n° 201 (n° spécial), p. 1.
es **Chemins de fer en Afrique du Nord** (Algérie, isie, Maroc et Méditerranée-Niger). (12 pages, nbreuses illustrations et cartes.)

)49 **656** (44)
re Métier, n° 202, 6 juin, p. 2.
AROIS. — La **concurrence rail-route.** (3 000 mots.)

)49 **656** .212.7 (44)
re Métier, n° 203, 13 juin, p. 3.
OUROCHE. — La **plus grande halle mécanisée** en struction à Paris-Tolbiac. (1 500 mots & fig.).

)49 **621** .431.72 (73)
re Métier, n° 205, 27 juin, p.3.
TEAD (A.L.). — Importance de la **traction Diesel** Etats-Unis. (1 500 mots & fig.)

L'Ossature métallique. (Bruxelles.)

1949 **656** .212.7 (44) & **725** .32 (44)
L'Ossature métallique, mars, p. 119.
REMOND (O.). — Les **Halles à marchandises** de Paris-Tolbiac. (4 000 mots & fig.)

1949 **624** .32 (42)
L'Ossature métallique, mars, p. 127.
Ponts métalliques récents en Grande-Bretagne. (1 500 mots & fig.)

1949 **624**
L'Ossature métallique, mars, p. 143.
DUTHEIL (J.). — **La conception des ossatures métalliques basée sur la déformation plastique.** (11 000 mots & fig.).

1949 **625** .13 (493)
L'Ossature métallique, avril, p. 173.
ROISIN (V.). — **Etançonnements métalliques** utilisés pour l'exécution du tunnel de la Jonction Nord-Midi. (2 000 mots & fig.)

1949 **624** .92 (493) & **656** .211.5 (493)
L'Ossature métallique, avril, p. 189.
PICALAUSA (P.). — La **charpente métallique soudée de la halle aux marchandises** sous voies de la nouvelle gare de Bruxelles-Midi. (3 000 mots & fig.)

1949 **624** .6
L'Ossature métallique, avril, p. 204.
TOTH (I.). — Calcul analytique des **arcs à deux articulations** à ligne moyenne circulaire. (2 500 mots, tableaux & fig.)

1949 **624**
L'Ossature métallique, mai, p. 263.
NICOLSKY (V.A.). — Contribution au **calcul des ponts curvilignes.** (1 100 mots & fig.)

Revue générale des Chemins de fer. (Paris.)

1948 **656** .211 (44) & **656** .212 (44)
Revue générale des Chemins de fer, décembre, p. 405.
DULAC & SANDOZ. — La **transformation des gares** de Paris-Sud-Est. (8 000 mots & fig.)

1948 **621** .7 (44)
Revue générale des Chemins de fer, décembre, p. 425.
BAUD (A.). — Les **ateliers « Traction électrique »** des Usines Alsthom à Belfort. (2 700 mots & fig.)

1948 **621** .133.7 (65)
Revue générale des Chemins de fer, décembre, p. 433.
SAMIE. — Le traitement interne intégral Armand des **eaux d'alimentation** des Chemins de fer Algériens. (1 000 mots & fig.)

1948 **621** .135.2
Revue générale des Chemins de fer, décembre, p. 436.
ARNAUD (P.). — **Tampons graisseurs à grand débit** permettant le roulement à grande vitesse sur boîtes d'essieux ordinaires. (1 000 mots & fig.)

1949 **625** .144.3
Revue générale des Chemins de fer, janvier, p. 1.
Le **raccordement parfait de M. A. Caquot,** de l'Académie des Sciences. (4 000 mots, tableau & fig.)

1949 **625.** 212 (44) & **625** .232 (44)
Revue générale des Chemins de fer, janvier, p. 9.
RIMBAUD & DUFOUR. — **Rames de voitures montées sur pneumatiques.** (6 000 mots & fig.)

1949 **621** .33 (44)
Revue générale des Chemins de fer, janvier, p. 19.
VANBOURDOLLE. — La situation actuelle des travaux d'**électrification** de la ligne Paris-Lyon. (2 000 mots & fig.)

1949 **656** .212 (44)
Revue générale des Chemins de fer, janvier, p. 24.
BEYNET & BINCAZ. — La **gare de Lyon-Guillotière.** (6 000 mots & fig.)

1949 **621** .336
Revue générale des Chemins de fer, janvier, p. 36.
RAOULT & CHAMINADE. — **Captation à grande vitesse** sous 1 500 V. du courant continu de traction. (3 600 mots & fig.)

1949 **625** .151 (44)
Revue générale des Chemins de fer, janvier, p. 49.
Le **branchement tg 0.03** de la S. N. C. F. (800 mots & fig.)

Revue des Transports et des Communications. (Lake Success.)

1949 **385** .62 & **385** .63
Revue des transports et des communications, janvier-mars, p. 10.
Les **Conventions de Berne** et l'**Office Central des Transports internationaux par Chemins de fer.** (6 000 mots.)

1949 **656** .235 (44)
Revue des transports et des communications, janvier-mars, p. 30.
La **nouvelle tarification marchandises** des chemins de fer français. (5 000 mots & fig.)

1949 **385** (73)
Revue des transports et des communications, avril-juin, p. 17.
HENRY (R.S.). — Les **chemins de fer** aux Etats-Unis. (16000 mots.)

In German.

Glasers Annalen. (Berlin.)

1949 **621** .392 & **625** .24
Glasers Annalen, März, S. 45.
KLOUGT (W). — Rückblick und Ausblick in die **Schweisstechnik des Waggonbaues.** (4 500 Wörter.)

1949 **625**
Glasers Annalen, März, S. 48.
BREUER (M.). — Neue Aufgaben für die **elektri Zugbeleuchtung.** (4 200 Wörter & Abb.)

1949 **625**
Glasers Annalen, März, S. 52.
Entwurf vierachsiger Personenzug-Wagen für Ha strecken und stark belastete Nebenstrecken. (3 Wörter, Tafel & Abb.)

1949 **656** .2
Glasers Annalen, März, S. 56.
Eisenbahnwagenkipper. (700 Wörter & Abb.)

1949 **656**
Glasers Annalen, April, S. 59.
NORDMANN (Dr. Ing. H.). — Neue Beiträge **Kenntnis des Zugwiderstandes.** (8 000 Wörter, Tab & Abb.)

1949 **621** .131.2
Glasers Annalen, April, S. 75.
MECKEL (A.). — Die Fortführung des **Lokom Normenwerkes.** (5 000 Wörter.)

1949 **621** .133.3 & **621** .
Glasers Annalen, Mai, S. 85.
RICHTER (H.). — **Verringerung des Betriebsaus von Dampflokomotiven durch vorbeugende Kesselpf** (2 000 Wörter & Abb.)

Signal und Draht. (Frankfurt-Main.)

1949 **656** .25
Signal und Draht, Nr. 4, April, S. 29.
DOBMAIER (A.). — **Signal- und Fernmeldewesc** Vereinigten Wirtschaftsgebiet 1948/49. (11 000 Wö

1949 **656** .251
Signal und Draht, Nr. 4, April, S. 38.
CHAUSSETTE (G.). — Die **Signalflügelstellung** den englischen Eisenbahnen. (2 000 Wörter.)

1949 **656**
Signal und Draht, Nr. 5, Mai, S. 45.
ELTZNER (C.). — **Schilder und Signale mit Em schmelzüberzug.** (1 300 Wörter.)

In English.

Bulletin, American Railway Engineerin Association. (Chicago.)

1949 **625** .143
Bulletin, American Railway Engineering Associa No. 479, February, pp. 489, 506, 508, 514 and Report of Committee : 4. — Rail. — **Rail f statistics** (2 300 words & tables). 5. — **Economic of various sizes of rail.** 6. — **Continuous welde** (1 200 words, fig. & tables). 8. — **Joint bar wea failures; revision of design and specification for new for maintenance repairs** (2 000 words & tables). 9. — **rugated rail : causes and remedy** (600 words).

1949 **625** .14 (73)
ulletin, American Railway Engineering Association, No. 479, February, p. 567.
Report of Committee : 5. — Track. — **Hold-down astenings for tie plates, including elastic pads under lates; their effect on tie wear.** (6 000 words, tables & g.)

The Engineer. (London.)

1948 **625** .142.4 (42)
he Engineer, December 31, p. 681.
A concrete sleeper making plant at a Royal Ordnance actory. (1 600 words & fig.)

1949 **385** (09 (42)
he Engineer, December 31, p. 7; January 14, p. 36.
British Railways in 1948. (3 600 words & fig.)

1949 **621** .438 (42)
he Engineer, December 31, p. 25.
Gas turbine locomotives for British Railways. (1 400 words & fig.)

1949 **625** .2 (42)
he Engineer, December 31, p. 27.
Rolling stock standardisation on British Railways. 500 words.)

1949 **656** .254 (42)
he Engineer, January 21, p. 73.
Re-signalling at Doncaster. (3 000 words & fig.)

Engineering. (London.)

1948 **624** .32 (73)
ngineering, No. 4324, December 10, p. 553.
Railway bridge over the Colorado river. (1 200 words & fig.)

1948 **621** .335 (68)
ngineering, No. 4324, December 10, p. 576.
3 000 H.P. Electric locomotive for South African Railways. (100 words.)

1948 **385** (09 (42)
ngineering, No. 4327, December 31, p. 641.
British Railways mechanical and electrical engineering reorganisation. (600 words.)

Journal of the Institute of Transport. (London.)

1949 **656** .225 (42)
Journal of the Institute of Transport, May, p. 117.
HAVERS (E.). — **Development in rail freight facilities.** (1 600 words & fig.)

Modern Transport. (London.)

1949 **621** .431.72 (42)
Modern Transport, May 14, p. 3.
A new Diesel shunter. (700 words & fig.)

1949 **625** .173 (42)
Modern Transport, May, 14, p. 15.
Track relaying. New machines for Western Region. (600 words & fig.).

1949 **625** .232 (42)
Modern Transport, May, 21, p. 15; July 2, p. 15.
Modern railway carriages *(To be continued.)* (1 800 words & fig.)

1949 **625** .4 (485)
Modern Transport, June 4, p. 3.
Construction of the Stockholm underground. (1 200 words & fig.)

1949 **656** .211.4 (73)
Modern Transport, June 4, p. 12; June 11, p. 6; July 16, p. 5.
POULTNEY (E.C.). — **Modern U. S. A. locomotive terminals.** (3 000 words & fig.)

Proceedings, American Society of Civil Engineers. (New York.)

1948 **656** .236
Proceedings, American Society of Civil Engineers, November, p. 1427.
HALL. (C.L.). — **The logical system of freight rates.** (3 500 words.)
COURLAND (R.H.). — **Discussion.** (400 words.)

1949 **624** (0
Proceedings, American Society of Civil Engineers, February, p. 275.
GRAVINA (P.B.J.) and RAMASWAMY (G.S.). — A simplified method of analyzing **suspension bridges.** (1 600 words.)

1949 **721** .9
Proceedings, American Society of Civil Engineers, April, p. 441.
BLANKS (R.F.). — **Modern concepts applied to concrete aggregate.** (14 000 words, tables & fig.)

1949 **624** .5
Proceedings, American Society of Civil Engineers, April, p. 489.
HUI-SIEN-LIU. — **A simplified method of analyzing suspension bridges.** (2 000 words.)

Railway Age. (New York.)

1949 **625** .214 (73) & **621** .431.72 (73)
Railway Age, May 7, p. 53.
BARR (W.M.). — **Rolling stock lubrication problems.** (2 200 words & fig.)

1949 **656** .223 (73)
Railway Age, May 14, p. 38.
SPOWART (D.J.). — **Car records on tabulating equipment.** (1 000 words, tables & fig.)

1949 **656** .212 (73)
Railway Age, May 14, p. 50.
Special « pits » aid freight car inspection. (1 200 words & fig.)

1949 **625** .13 (73) & **625** .14 (73)
Railway Age, May 28, p. 22.
Major line relocation nears completion. (2 000 words & fig.)

1949 **625** .232 (73)
Railway Age, May 28, p. 46.
C. B. and Q. renovates «Denver zephyrs». (500 words & fig.)

1949 **621** .431.72 (73)
Railway Age, June 4, p. 32.
Davenport-Besler builds **three-car Diesel train** for Colombia. (2 200 words & fig.)

1949 **625** .243 (73)
Railway Age, June 4, p. 40.
New car carries bulk-loaded flour. (300 words & fig.)

Railway Engineering and Maintenance. (Chicago.)

1948 **625** .173 (73)
Railway Engineering and Maintenance, December, p. 1271.
Two multiple tampers used on track-raising job. (1 800 words & fig.)

1948 **625** .173 (73)
Railway Engineering and Maintenance, December, p. 1274.
VISINTAINER (A.A.). — **Special technique simplifies this cover-plate renewal job.** (1 900 words & fig.)

1948 **625** .173 (73)
Railway Engineering and Maintenance, December, p. 1278.
Imported machine cleans the entire ballast section. (1 000 words & fig.)

Railway Gazette. (London.)

1948 **385** .4 (42)
Railway Gazette, No. 26, December 24, p. 719.
Mechanical and electrical engineering organisation, British Railways. (600 words & diagram).

1948 **656** .255 (945)
Railway Gazette, No. 26, December 24, p. 724.
Single-line working on the Victorian Railways. (1 800 words & fig.)

1948 **621** .335
Railway Gazette, No. 27, December 31, p. 749.
Power bogie transmission gears. (600 words & fig.)

1948 **313** .385 (42)
Railway Gazette, No. 27, December 31, p. 758.
British Transport Commission Statistics. Summary of the principal statistics for the four week period ended October 31. (600 words & tables.)

1949 **625** .13 (43)
Railway Gazette, No. 1, January 7, p. 17.
Reconstruction of Rhine bridge at Düsseldorf. (700 words & fig.)

1949 **625** .2 (4
Railway Gazette, No. 1, January 7, p. 23.
Rolling stock standardisation, British Railways. (5
words.)

1949 **621** 438 (4
Railway Gazette, No. 2, January 14, p. 39.
Experimental gas turbine locomotives, Western Regio
(1 100 words & fig.).

1949 **656** .281 (4
Railway Gazette, No. 2, January 14, p. 50.
Ministry of Transport Accident Report. Near Wa
Road Junction : British Railways, London Midla
Region : May 18, 1948. (1 800 words & fig.)

1949 **625** .13 (5
Railway Gazette, No. 3, January 21, p. 71.
Tunnel widening on the Thull Ghat, G. I. P. R. (8
words & fig.)

1949 **625** .4 (48
Railway Gazette, No. 3, January 21, p. 72.
The Progress of Stockholm Underground (500 word

1949 **621** .132.1 (4
Railway Gazette, No. 3, January 21, p. 73.
Converted Brighton Atlantic locomotive. (300 wor
& fig.)

1949 **656** .283 (4
Railway Gazette, No. 3, January 21, p. 78.
Ministry of Transport Accident Report : Coningt
North, British Railways, Eastern Region, March 1 a
October 16, 1948. (1 800 words.)

1949 **621** .335 (494) & **656** .222.1 (4
Railway Gazette, No. 4, January 28, p. 99.
ALLEN (Cecil J.). — **Modern Swiss electric loc
motive performance.** (800 words & tables.)

1949 **621** .338 (4
Railway Gazette, No. 4, January 28, p. 100.
Aluminium alloy rolling stock for London Transpo
(400 words.)

1949 **621** .134.1 (4
Railway Gazette, No. 4, January 28, p. 101.
**Measuring crank-pin settings on locomotive coupl
wheels.** (300 words & fig.)

1949 **621** .33 (4
Railway Gazette, No. 4, January 28, p. 109.
Electrification and Diesel-electrification in Holla
(600 words.)

1949 **625** .143.1 (4
Railway Gazette, No. 4, January 28, p. 124.
The new standard British rail. (800 words & fig.)

1949 **625** .11
Railway Gazette, No. 4, January 28, p. 128.
BULKELEY (G.V.O.). — **A railway lifeline acr
Africa.** (400 words & map.)

1949 **621** .95 (4
Railway Gazette, No. 4, January 28, p. 129.
New two-spindle rail-drilling machine. (200 wo
& fig.)

1949 **625** .174 (42)
ailway Gazette, No. 4, January 28, p. 129.
Clearing snow on British Railways. (200 words & fig.)

1949 **625** .144.1 (68)
ailway Gazette, No. 6, February 11, p. 154.
Long rails in South Africa. (300 words.)

1949 **625** .143.3 (944)
ailway Gazette, No. 6, February 11, p. 157.
A rail flow detector car. (300 words & fig.)

1949 **656** .281 (42)
ailway Gazette, No. 6, February 11, p. 161.
Ministry of Transport Accident Report. New Southgate, astern Region, British Railways : July 17, 1948. (200 ords & fig.)

1949 **385** .32 (42)
ailway Gazette, No. 7, February 18, p. 179.
MILES BEEVOR. — **Organisation of the British ransport Commission.** (3 000 words.)

1949 **621** .135 (01 (42)
ailway Gazette, No. 7, February 18, p. 183.
THOMPSON FAIRLESS. — **Rebalancing three-linder locomotives.** (1 200 words & fig.)

1949 **621** .331 (42)
ailway Gazette, No. 8, February 25, p. 209.
Frequency correcting equipment. Southern Region. 00 words & fig.)

1949 **656** .211.5 (42)
ailway Gazette, No. 8, February 25, p. 217.
Modernised ticket office at Bowes Park, Eastern egion. (600 words & fig.)

1949 **313** .385 (42)
ailway Gazette, No. 8, February 25, p. 218.
British Transport Commission Statistics. (200 words tables.)

1949 **625** .231 (42)
ailway Gazette, No. 9, March 4, p. 245.
Experimental British double-deck train. (400 words & .)

1949 **621** .132.1 (73)
ailway Gazette, No. 9, March 11, p. 264.
POULTNEY (E.C.). — **Modern motive power on e C. & O.** (2 400 words & fig.)

1949 **625** .143.1 (42)
ailway Gazette, No. 9, March 11, p. 269.
British flat-bottom track. (100 words & fig.)

1949 **621** .438
ailway Gazette, No. 9, March 11, p. 273.
Gas turbine locomotives. (1 800 words & fig.)

1949 **621** .132.1 (6)
ailway Gazette, No. 10, March 18, p. 292.
British built locomotives for Portuguese West Africa. 200 words & fig.)

1949 **621** .431.72 (73) & **625** .28 (73)
ailway Gazette, No. 10, March 18, p. 295.
Design for a **high-speed lightweight train.** (500 words fig.)

1949 **656** .285 (42)
Railway Gazette, No. 10, March 18, p. 302.
Ministry of Transport Accident Report : Ardler Junction, Scottish Region, British Railways : July 17, 1948. (1 000 words & fig.)

In Spanish.

Ferrocarriles y Tranvias. (Madrid.)

1949 **625** .251
Ferrocarriles y Tranvias, abril, p. 125.
CERNUDA (R. J.). — **El frenado de los trenes de viajeros** entre los 70 y 140 kilómetros por hora. (1 500 palabras.)

1949 **385** & **623**
Ferrocarriles y Tranvias, abril, p. 136.
GIOVANELLI (J. A.). — **Importancia de los ferrocarriles en las dos últimas guerras.** (10 000 palabras.)

1949 **621** .131
Ferrocarriles y Tranvias, mayo, p. 174; julio, p. 255.
MARTIN SARRALDE (A.). — Nótas referentes a **locomotoras de vapor.** (4 000 palabras & fig.)

1949 **625** .143.5
Ferrocarriles y Tranvias, mayo, p. 189.
Escarpia elástica para carriles. (500 palabras & fig.)

In Italian.

Giornale del Genio Civile. (Roma.)

1949 **624** .6
Giornale del Genio Civile, maggio, p. 207.
GRAVINA (P. B. J.). — Sul **calcolo statico dei ponti ad arco** sottile con trave irrigidente secondo la teoria del II ordine. (4 000 parole & fig.)

Ingegneria ferroviaria. (Roma.)

1949 **656** .257 (45)
Ingegneria ferroviaria, maggio, p. 319.
BELLOMI (C.). — **Evoluzione degli apparati centrali** sulla rete delle F. S. (8 000 parole & fig.)

1949 **656** .256.3 (73)
Ingegneria ferroviaria, maggio, p. 333.
JACHINO (C.). — **Codice americano dei segnali ferroviari.** (5 000 parole & fig.)

1949 **656** .253 (494)
Ingegneria ferroviaria, maggio, p. 341.
DE JOB (E.). — **Nuovo segnalamento** delle Ferrovie Federali Svizzere. (3 000 parole & fig.)

1949 **656** .254 (492)
Ingegneria ferroviaria, maggio, p. 361.
VERSTEGEN (J. H.). — **Segnali a funzionamento automatico per passaggi a livello** incustoditi sulle Ferrovie Olandesi dello Stato. (1 500 parole & fig.)

Trasporti Pubblici. (Roma.)

1949 **625** .231 (45)
Trasporti Pubblici, febbraio, p. 154.
Nuovo materiale rotabile delle Ferrovie dello Stato. (1 500 parole & fig.)

1949 **656** .231 (45)
Trasporti Pubblici, febbraio, p. 157.
La **riforma tariffaria** delle Ferrovie dello Stato. (1 000 parole.)

In Polish. (= 91.885)

Przeglad Kolejowy. (Warsaw.)

1949 **385** (09.3 (437) = 91.885
Przeglad Kolejowy, No. 1, p. 3; No. 2, p. 34.
JANKOWSKI (J.). — **Centenary of railways** on Polish territories. (11 000 words & fig.)

1949 **656** .256 = 91.885
Przeglad Kolejowy, No. 1, p. 16.
BARYSZ (E.). — Analytical method of calculation of **block distance on double track railways.** (5 000 words & fig.)

1949 **625** .14 (01 = 91.885
Przeglad Kolejowy, No. 2, p. 42; No. 3, p. 66.
JACYNA (W.). — On the **base coefficient of elasticity (C) of the superstructure of railway track** with reference to Prof. Watsutynski's researches. (10 000 words & fig.)

1949 **625** .13 = 91.885
Przeglad Kolejowy, No. 3, p. 91.
KIELKIEWICZ (W.). — **Ventilation of railway tunnels.** (3 000 words.)

1949 **385** .114 = 91.885
Przeglad Kolejowy, No. 4, p. 101.
BESSAGA (M.). — Influence of certain operative factors on the **unit cost for transport.** (2 000 words, tables & fig.)

1949 **621** 431.72 (437) = 91.885
Przeglad Kolejowy, No. 4, p. 120.
WISZNIEWSKI (T.). — **Guiding principles for the construction of the electric motorcoaches** of the Polish State Railways. (4 000 words & fig.)

Przeglad Komunikacyjny. (Warsaw.)

1949 **656** = 91.885
Przeglad Komunikacyjny, No. 1, p. 2.
LOPUSZYNSKI (M.). — **Planning of communications.** (12 000 words.)

1949 **656** = 91.885
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Zur Entwicklung der Diesel-Lokomotiven und Die Triebwagen in Deutschland. (14 000 Wörter, Tafel Abb.)

1949 **62**
Glasers Annalen, August, S. 133.
HEMPEL (M.) & SANDER (H. R.). — **Festigke eigenschaften** unter Zugbelastung mit verschiede Belastungsgeschwindigkeiten. (2 500 Wörter & A

Signal und Draht. (Frankfurt-Main.)

1949 **656** .25
Signal und Draht, Nr. 6, Juni, S. 53.
JAEGER (K.). — Betrachtungen über das r **Signalsystem.** (4 000 Wörter & Abb.)

1949 **656** .254
Signal und Draht, Nr. 6, Juni, S. 58; Nr. 8, Aug S. 77.
BREKLE (O.). — Die **Wirtschaftlichkeit der F meldeanlagen** für Bau, Betrieb und Verkehr der Rei bahn. (6 000 Wörter & Abb.)

1949 **656**
Signal und Draht, Nr. 7, Juli, S. 65.
CHRISTOPH. — Über **Lichtsignale.** Betrachtur und Vorschläge zu einer neuen Signalordnung ortsfeste Lichtsignale. (3 000 Wörter.)

1949 **656**
Signal und Draht, Nr. 8, August, S. 79.
KUNZE (H.). — **Signalgebung und Fahrdyna** (1 200 Wörter & Abb.)

1949 **656**
Signal und Draht, Nr. 9, September, S. 95.
WARLICH (G.). — **Eingriffe bei Störungen in H blockanlagen.** (2 500 Wörter.)

1949 **656** .257
Signal und Draht, Nr. 10, Oktober, S. 101.
REINHARD (K.). — **Relais-Stellwerke** in Schwe (4 000 Wörter & Abb.)

In English.

Bulletin, American Railway Engineering Association. (Chicago.)

1949 **625** .142
lletin, American Railway Engineering Association, June-July, p. 29.
CODE (C. J.) and MARTIN (E. E.). — **Test of three thods of attaching tire plates.** (6 000 words, tables & fig.)

The Engineer. (London.)

1949 **625** .4 (42)
e Engineer, No. 4858, March 4, p. 234.
DAWSON (O.). — **Gants Hill Station,** London ansport. (2 400 words & fig.)

1949 **621** .335 (492) & **621** .431 .72 (492)
e Engineer, No. 4858, March, 4, p. 255.
VAN LESSEN (H. J.). — **Electric and Diesel-electric action** on the Netherlands Railways. (3 000 words fig.)

1949 **621** .335 (42)
e Engineer, No. 4859, March, 11, p. 280.
Southern Region **electric locomotive** No. 20003. (2 400 rds & fig.)

1949 **625** .26 (42)
e Engineer, No. 4860, March, 18, p. 298.
Rolling stock overhaul at Acton Works. (2 000 words fig.)

1949 **621** .131 .2 (42)
e Engineer, No. 4861, March 25, p. 318.
MAC MILLAN (R. H.). — **Epicyclic gear trains.** 200 words & fig.)

1949 **625** .123 (42)
e Engineer, No. 4861, March 25, p. 324.
Drainage of permanent way at Hougham. (800 words fig.)

1949 **721** .9
gineer, No. 4867, May 6, p. 496.
WILLIAMS (A. E.). — **Refractory concrete.** (2 000 rds & fig.)

1949 **621** .232 (42)
gineer, No. 4867, May 6, p. 507.
All-steel fabricated railway coaches. (600 words & fig.)

Engineering. (London.)

1949 **691**
gineering, No. 4331, January 28, p. 89.
Thermic drilling of concrete and stone. (2 000 words fig.)

1949 **698** (42)
gineering, No. 4331, January 28, p. 91.
RIGG (J. G.) & SKERREY (E. W.). — **Priming nts for light alloys.** (1 800 words.)

1949 **625** .143 .1 (42)
Engineering, No. 4332, Feb. 4, p. 113.
Standard flat-bottom rails for British Railways. (800 words & fig.)

1949 **625** .142 .4 (42)
Engineering, No. 4333, Febr. 11, p. 138.
The construction of **prestressed concrete railway sleepers.** (1 600 words & fig.)

Journal, Permanent Way Institution. (London.)

1949 **625** .174 (42)
Journal, Permanent Way Institution, August, p. 112.
PATERSON (W.). — **Winter and Railways.** (3 200 words & fig.)

1949 **625** .173 (42)
Journal, Permanent Way Institution, August, p. 121.
WEST (J. D.). — **The Southern Region track laying machine.** (1 800 words & fig.)

1949 **665** .882 (42)
Journal, Permanent Way Institution, August, p. 126.
WALTERS (D. J.). — **Oxy-acetylene cutting.** (1 400 words & fig.)

1949 **625** .14 (43)
Journal, Permanent Way Institution, August, p. 130.
ORCHARD (H. C.). — **Trackwork on the Reichsbahn.** (3 000 words.)

The Locomotive, Railway Carriage & Wagon Review. (London.)

1949 **621** .438 (73)
The Locomotive, September 15, p. 131.
Alco-G. E. gas turbine loco. (1 200 words & fig.)

1949 **621** .335 (494)
The Locomotive, September 15, p. 135.
HAUT (F. J. G.). — **Swiss electric locomotives.** (2 000 words & fig.)

1949 **621** .335 (73)
The Locomotive, September 15, p. 143.
New electric locomotives, Virginian Railroad. (200 words & fig.)

Modern Transport. (London.)

1949 **625** .13 (42)
Modern Transport, July 9, p. 7.
New Woodhead tunnel. (600 words & fig.)

1949 **385** .3 (54)
Modern Transport, July 9, p. 8.
Indian State Railways. Report by Board for 1947-1948. (800 words.)

1949 **625** .162 (42)
Modern Transport, July 9, p. 12.
Level crossing accidents. (800 words.)

1949 **625** .4 (485)
Modern Transport, July 30, p. 3.
Progress of **the Stockholm underground.** (1 200 words & fig.)

1949 **621** .132 .8 (42)
Modern Transport, July 30, p. 5.
POULTNEY (E. C.). — **Landmarks of express locomotive progress.** — 13) Boiler steam pressures *(to be continued)*. (1 800 words, tables & fig.)

1949 **625** .617 (44)
Modern Transport, July 30, p. 9.
New French designs for containers. (2 000 words & fig.)

The Oil Engine and Gas Turbine. (London.)

1949 **621** .431 .72 (42)
The Oil Engine and Gas Turbine, May, p. 12.
Locomotive with reversible engine. (800 words & fig.)

1949 **621** .431 .72 (42) & **625** .285 (42)
The Oil Engine and Gas Turbine, May, p. 14.
British Diesel rail vehicles for export. (900 words & fig.)

1949 **621** .132 .7 (42)
The Oil Engine and Gas Turbine, June, p. 44.
Locomotives designed for dock duties. (1 200 words & fig.)

1949 **621** .431 .72 (42)
The Oil Engine and Gas Turbine, June, p. 56.
Twin locomotives' non-stop runs. (1 000 words & fig.)

1949 **621** .438 (494)
The Oil Engine and Gas Turbine, June, p. 58.
On the footplate of the **Swiss gas turbine locomotive.** (1 500 words, tables & fig.)

Railway Age. (New-York.)

1949 **656** .254 (73)
Railway Age, June 11, p. 36.
C. T. C. saves train time on Frisco. (1 500 words & fig.)

1949 **625** .232 (73)
Railway Age, June 18, p. 44.
B. & O. puts new « Columbian » in service. (3 000 words & fig.)

1949 **656** .254 (73)
Railway Age, June 18, p. 52.
CORPORON (P. J.). — **Radio freed from wire-line linkage.** (1 200 words & fig.)

1949 **656** .212 .6 (73)
Railway Age, June 18, p. 56.
Overhead conveyor expedites freighthouse operations. (1 000 words & fig.)

1949 **625** .244
Railway Age, June 25, p. 74.
Refrigerator car research progresses. (2 400 w & fig.)

Railway Engineering and Maintenance. (Chicago.)

1949 **625** .14
Railway Engineering and Maintenance, April, p. 3
PROTZELLER (H. W.). — **Track stabilizatio scientific approach - Part. I.** (1 800 words & fig.)

1949 **625** .173
Railway Engineering and Maintenance, April, p. 3
HURSH (S. R.). — **Mechanization the answer to costs.** (1 200 words & fig.)

1949
Railway Engineering and Maintenance, April, p. 3
LAIRD (A. N.). — **Overcoming failures in ma structures.** (1 400 words & fig.)

1949 **625** .14 (01
Railway Engineering and Maintenance, May, p. 4
PROTZELLER (H. W.). — **Track stabilizatio scientific approach.** (2 100 words & fig.)

Railway Engineering and Maintenance, May, p. 4
MEYER (F. G.). — **Tests Penta treatment for by hot and cold bath.** (800 words, tables & fig.)

Railway Gazette. (London.)

1949 **621** .431 .72
Railway Gazette, No. 23, June, 10, p. 650.
Trial run of L. M. R. twin-unit Diesel. (500 words &

1949 **621** .33
Railway Gazette, No. 24, June 17, p. 669.
Express electric sets in Sweden. (1 000 words &

1949 **621** .133 .1
Railway Gazette, No. 24, June 24, p. 697.
Mixed-traffic locomotives for Egypt. (600 words &

1949 **625** .13
Railway Gazette, No. 24, June 17, p. 696.
Removing a bottleneck on the Gotthard line. (400 wo

1949 **621** .431 .72
Railway Gazette, No. 1, July 1, p. 17.
British Railways, new Diesel locomotive. (300 v & fig.)

1949 **62**
Railway Gazette, No. 2, July 8, p. 40.
BULKELEY (G. V. O.). — **Motive power requirer of Colonial Railways.** Advantages of steam and d electric motive power under the special conditio Colonial railways. (1 100 words.)

949 **621** .35 (42)
lway Gazette, No. 2, July 8, p. 46.
oll-out battery box for railway carriages. (800 words ig.)

949 **313** .385 (42)
lway Gazette, No 2, July 8, p. 50.
ritish Transport Commission Statistics. Summary he principal statistics for the four-week period ended il 24. (400 words & tables.)

949 **621** .33 (492)
lway Gazette, No. 3, July 15, p. 68.
letherlands Railways electrification extension. (400 ds & map.)

949 **621** .335 (44)
lway Gazette, No. 3, July 15, p. 69
lotive power for Paris-Lyons electrification. (1 200 ds & fig.)

949 **625** .232 (73)
lway Gazette, No. 3, July 15, p. 72.
otating-reclining seats on the Pennsylvania Railroad.) words & fig.)

949 **625** .244 (6)
lway Gazette, No. 3, July 15, p. 73.
cold store installation in East Africa. (600 words & fig.)

949 **388** (42)
lway Gazette, No. 4, July 22, p. 97; No. 28, August 5, p. 151.
lew railways for London. (3 000 words & maps.)

949 **621** .132 .5 (44)
lway Gazette, No. 4, July 22, p. 101.
Experimental **2-12-0 locomotive** for the French National lways. (1 100 words & fig.)

949 **656** .211 (45)
lway Gazette, No. 5, July 29, p. 124.
lew passenger station in Rome. (800 words & fig.)

949 **313** .385 (42)
lway Gazette, No. 5, July 29, p. 134.
British Transport Commission Statistics. (Four week iod ended May 22). (300 words & tables.)

949 **621** .33 (494)
lway Gazette, No. 28, August, 5, p. 152.
he Rhaetian Railway. (4 000 words, tables, maps ig.)

The Railway Magazine (London.)

949 **621** .132 .1 (42)
Railway Magazine, May-June, p. 153.
ocomotive design on the Southern Railway. (2 000 ds & fig.)

949 **621** .438 (42)
Railway Magazine, May-June, p. 185.
Gas turbine locomotives. (1 500 words.)

1949 **621** .232 (42)
The Railway Magazine, May-June, p. 193.
LEE (Ch. E.). — **Double-deck railway carriages.** (800 words & fig.)

1949 **625** .232 (42)
The Railway Magazine, July and August, p. 255.
British Railways all steel carriages. (1 000 words & fig.)

Railway Mechanical Engineer. (New York.)

1948 385 (061 .4 (73)
Railway Mechanical Engineer, August, p. 59.
Association of American Railroads. Mechanical division annual meeting at Chicago. (39 pages.)

1948 621 .431 .72
Railway Mechanical Engineer, August, p. 102.
LAMBORN (R.). — **The locomotive traction generator comes of age.** (2 800 words, tables & fig.)

1948 **656** .211 .4 (73) & **656** .215 (73)
Railway Mechanical Engineer, August, p. 107.
Power and light for terminals. (2 000 words & fig.)

1948 **625** .26 (73)
Railway Mechanical Engineer, September, p. 61.
N. & W. rip. track spot system. (1 600 words & fig.)

1948 **621** .138 .5 (73) & **621** .431 .72 (73)
Railway Mechanical Engineer, September, p. 65.
New Haven Diesel repair shop. (2 200 words & fig.)

1948 **625** .236 (73)
Railway Mechanical Engineer, September, p. 72.
Burlington car-roof washer. (1 800 words & fig.)

1948 **621** .431 .72 (73)
Railway Mechanical Engineer, September, p. 75.
BROWN (C. W.) Jr. and DRAWBAUGH (D. G.) Jr. — **Treatment and control of Diesel radiator water.** (1 200 words & fig.)

1948 **621** .431 .72 (73)
Railway Mechanical Engineer, September, p. 95.
LAFFIN (D.). — **Reconditioning worn traction motor housings.** (2 200 words & fig.)

In Danish. (= 439.81)

Sikringsteknikeren (Copenhagen)

1949 **656** .257 (73) = 439.81
Sikringsteknikeren, N° 1 & 2, p. 377.
HANSEN (W.). — **Safety installations in U. S. A.** (Account of a journey). (2 500 words & fig.)

In Spanish.

Ferrocarriles y Tranvias. (Madrid.)

1949 **621** .131.
Ferrocarriles y Tranvias, julio, p. 249.
COSTILLA (B.). — Mas sobre **circulacion.** (3 000 palabras & fig.)

1949 **385**
Ferrocarriles y Tranvias, julio, p. 266.
NEGRETE GARCIA (C.). — **El ferrocarril en la paz y en la guerra.** (6 000 palabras.)

1949 **625** .28 (460)
Ferrocarriles y Tranvias, agosto, p. 291.
SIMÓ (J.) & DE VILLOTA (M.). — **Análisis técnico de nuestros ferrocarriles y orientaciones futuras de las locomotoras de vapor.** (6 000 palabras & fig.)

1949 **625** .232
Ferrocarriles y Tranvias, agosto, p. 301.
PRATS TOMAS (J.). — Posible mejora a introducir en los **coches-camas** al colocarles hasta veinte cabinas individuales a cada uno. (3 500 palabras & fig.)

1949 **656** .281
Ferrocarriles y Tranvias, agosto, p. 306.
ROMAN (C.). — El **descarrilamiento por obstáculos en el carril.** (3 500 palabras & fig.)

In Italian.

Ingegneria ferroviaria. (Roma.)

1949 **656** .222 & **656** .25
Ingegneria ferroviaria, luglio-agosto, p. 473.
GUZZANTI (C.). — Il **distanziamento dei treni.** (5 000 parole & fig.)

1949 **621** .335 (45)
Ingegneria ferroviaria, luglio-agosto, p. 485.
d'ARBELA (A.). — Le **nuove locomotive a quattro assi motori Gr. E 424** delle Ferrovie dello Stato. (*Continua.*) (5 000 parole, tavole & fig.)

1949 **692**
Ingegneria ferroviaria, luglio-agosto, p. 521.
DEL ZANNA (A.). — Nuovi **attrezzi per la verifica dei piani e per la misurazione degli spessori.** (1 000 parole & fig.)

1949 **656** .25
Ingegneria ferroviaria, settembre, p. 563.
BELLOMI (C.) & RIGHI (R.). — **Gli interuttori a scatto negli impianti di segnalamento.** (6 000 parole, tavole & fig.)

1949 **625** .62 (45)
Ingegneria ferroviaria, settembre, p. 575.
PATRASSI (A.). — Nuovi **rilievi speciali del traffico tramviario** di Milano. (5 000 parole & fig.)

In Dutch.

Spoor- en Tramwegen. (Utrecht.)

1949 **656** .212 .5 (4
Spoor- en Tramwegen, Nr. 15, 28 Juli, p. 231; Nr. 1
11 Augustus, p. 249.
Het **moderne rangeeremplacement.** (Tergnier, Vil
neuve-St-Georges). (3 000 woorden & fig.)

1949 **656** .235
Spoor- en Tramwegen, Nr. 16, 11 Augustus, p. 241.
BUTIJN (J. A. A.). — Het vraagstuk van de **fra**
keering in het internationale goederenverkeer per spoe
weg. (2 500 woorden.)

1949 **656** .2
Spoor- en Tramwegen, Nr. 18, 8 September, p. 275.
BUTIJN (J. A. A.) & VAN NIMWEGEN (F. A. J.).
Het probleem van de **uitvalverdeling in het internationa**
goederenverkeer. (3 500 woorden & fig.)

1949 **625** .13 (42 + 4
Spoor- en Tramwegen, Nr. 19, 22 September, p. 29
Nr. 20, 6 October, p. 316; Nr. 21, 20 Octob
p. 341.
De **Kanaaltunnel** en de treinveerdiensten. (8 000 woord
& fig.)

1949 **625** .2
Spoor- en Tramwegen, Nr. 19, 22 September, p. 30
WERVERS (J. H.). — Het **transport van banane**
(2 500 woorden & fig.)

In Swedish. (= 439.71)

Meddelanden från Svenska Lokaltrafik Föreningen.

1949 **388** (485) & **625** .42 (4
Meddelanden från Svenska Lokaltrafik Förening
N° 3, p. 109.
SAMUELSON (S.). — **Rapid transit system**
Stockholm. (4 000 words & fig.)

Nordisk Järnbanetidskrift. (Stockholm.)

1949 **385** .812 (47.1) = 439
Nordik Järnbanetidskrift, No. 2, p. 25.
ARPOKARI (Y.). — **Administrative organisation**
the Finnish State Railways. (2 200 words.)

1949 **656** .25 = 439.71
Nordisk Järnbanetidskrift, No. 2, p. 42.
Where do signalling systems might increase **traf**
safety. (2 200 words.)

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Vol. XLVI. — No. 12. DECEMBER 1949

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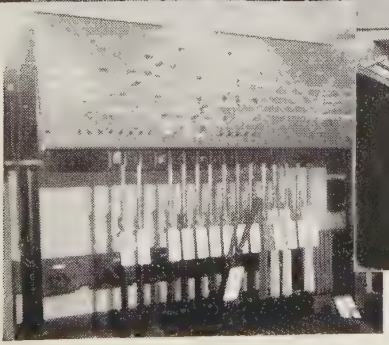

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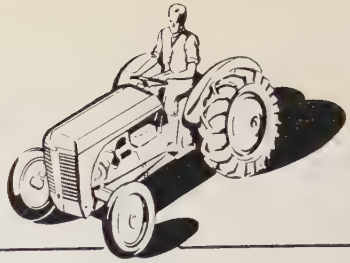

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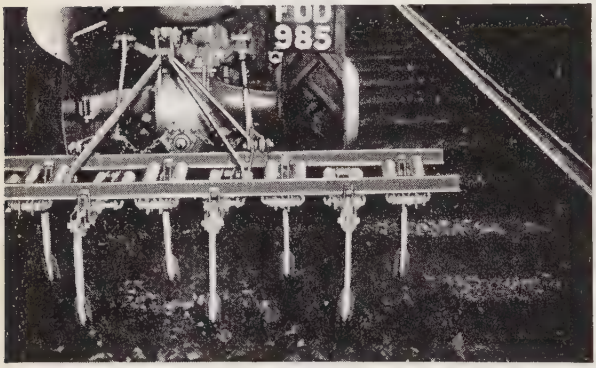

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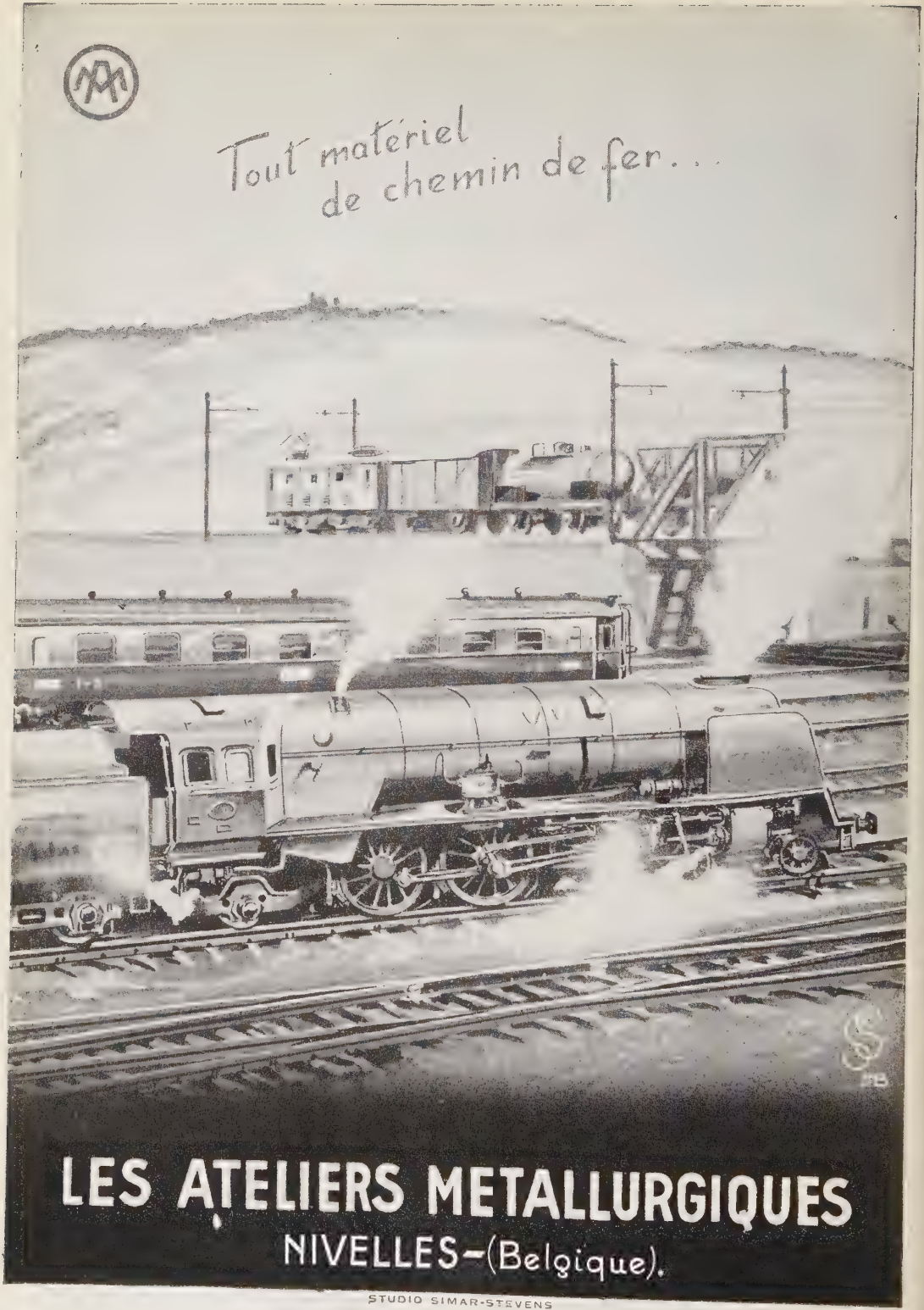
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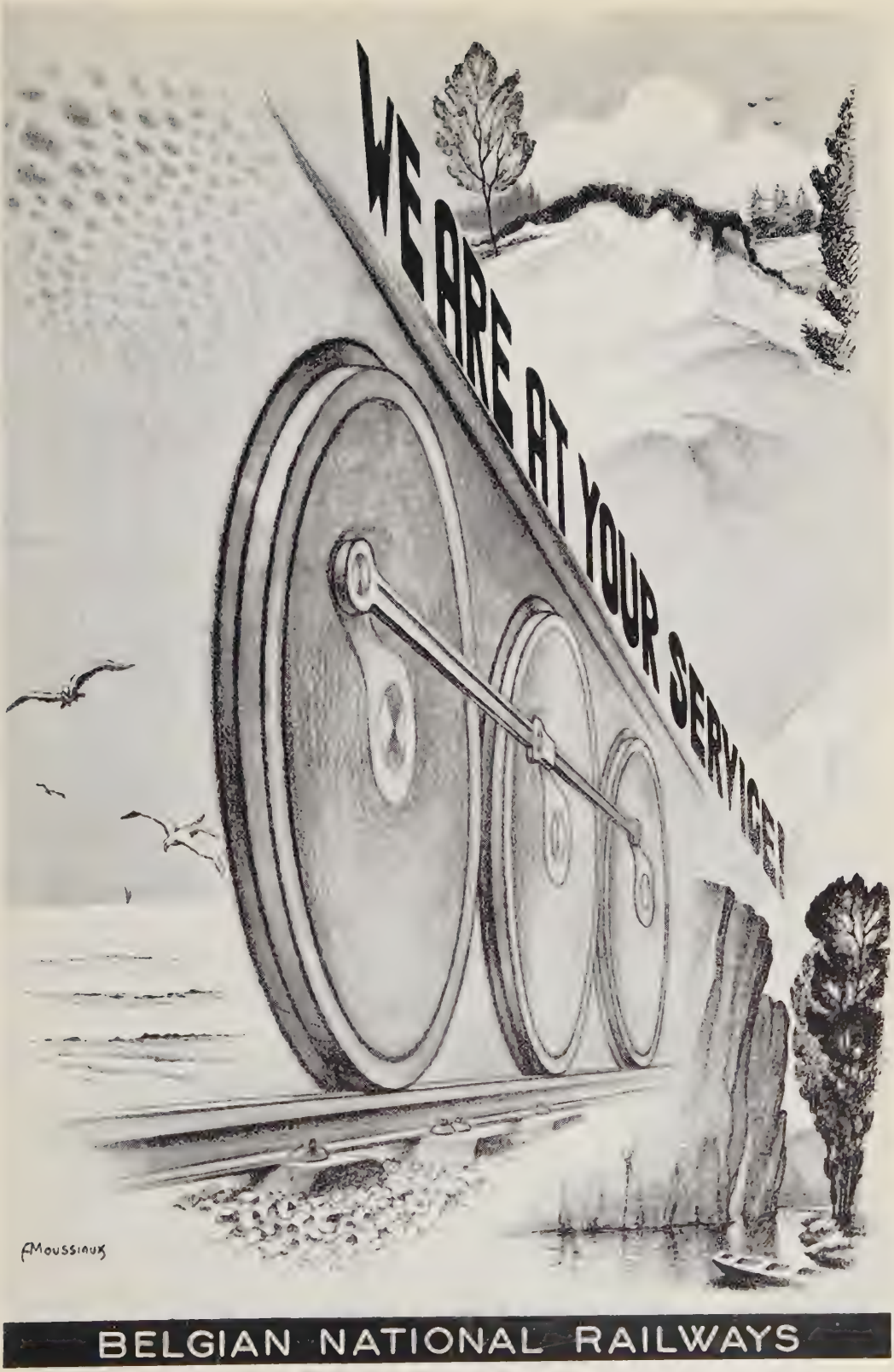
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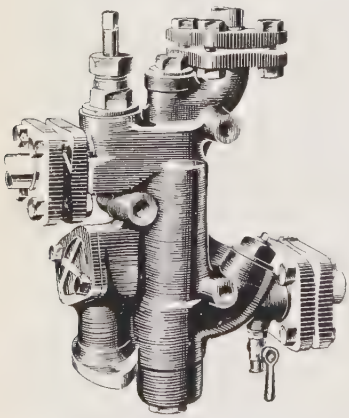

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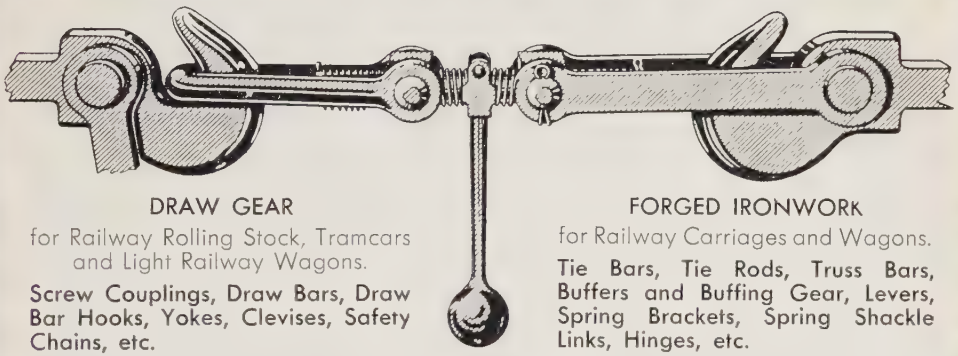

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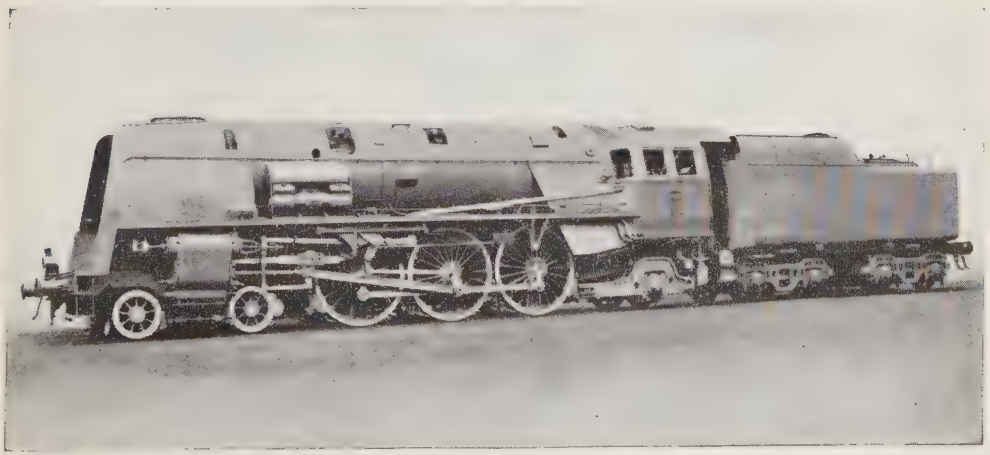

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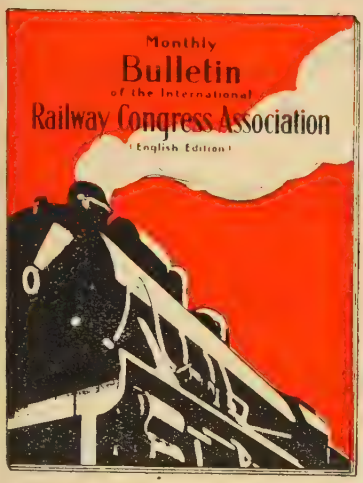

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